DETERMINATION OF THE EFFECTS OF URBANIZATION ON EXPECTED PEAK FLOWS FROM SMALL WATERSHEDS IN DeKALB COUNTY, GEORGIA

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DETERMINATION OF THE EFFECTS OF URBANIZATION

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SUMMARY

UROS: Urban/Rural Flood Simulation Model, developed at the Georgia Institute of Technology in 1974, simulates the hydrologic response of watersheds and provides well-founded estimates of expected peak flood flows. It was the purpose of this research to develop simplified equations that can be used on small watersheds (up to 200 acres) that closely approximate the estimated expected peak flood flows resulting from complete simulation with UROS. To generate these equations, simulations were made using a hypothetical watershed representative of the general physiography of DeKalb County. Varying degrees of urbanization were simulated considering the effects of impervious area and the effects of "sewering". For DeKalb County the effects of sewering were simulated as combinations of road gutters and hydraulically modified channels within the hypothetical watershed's natural channel system.

From the results of simulating various combinations of sewering and imperviousness within the hypothetical watershed, regression equations were generated for estimating the mean and standard deviation of the annual peak flow series that would be produced from complete simulation with UROS. Exact graphical solutions of these equations were prepared using the parameters drainage area, percent imperviousness, road density, and percent of channels hydraulically modified. Verification of the estimating accuracy of these equations was achieved through comparisons with simulated data from actual DeKalb County watersheds. Two-thirds of the estimated mean annual peak flow values were within 10 percent of the

corresponding simulated value. Similar results were achieved for estimating the standard deviation of the annual peak flow series. From this verification it is concluded that a simple method has been developed for accurately estimating the frequency-peak flow relationship that would result from UROS simulation of small watersheds in DeKalb County.

CHAPTER I

INTRODUCTION

Urbanization of a watershed creates hydrologic changes. Decreased base flow, increased volume of runoff, reduced watershed response time, and increased peak flow rates are four major effects which have received attention in previous research (James, 1965; Leopold, 1968; Anderson, 1968; Wallace, 1971). With respect to the increase in peak flow rates, research in urban hydrology has attempted to quantify this effect and has produced various engineering approaches ranging from purely empirical equations to numerous complex computer models. Because these several approaches produce significant variations in estimated peak flow rates, the practicing engineer finds himself faced with the dilemma of selecting a method which is commensurate with the scope of the problem, his expertise in the field of hydrology, the financial resources of his client, and occasionally the acceptability of selected procedures in a local jurisdiction. In one instance, the engineer may resort to very conservative estimates of flow values which can create high land development expenses. In another case, the engineer may rely on traditional calculating methods which have found acceptance through repeated use, even though in many cases they are illogically based and in serious want for substantial evidence of accuracy. A single acceptable and adequate method for estimating peak flow rates is desirable to resolve this dilemma.

In the fall of 1973, DeKalb County, Georgia through the Board of Commissioners, contracted with the School of Civil Engineering, Georgia

Institute of Technology for the development of a hydrologic simulation model calibrated to the specific hydrologic regime of DeKalb County (Lumb, 1975). The purpose of the model, known as UROS: Urban/Rural Flood Simulation Model, was to provide a comprehensive tool for estimating expected peak flows. The model avoids the selection of subjective coefficients and performs objective calculations in a consistent manner from one simulation to the next.

UROS routes precipitation excess from historical rainfall events. The precipitation excess has been previously determined for several representative soils of DeKalb County by continuous simulation of the hydrologic cycle using the Georgia Tech Watershed Simulation Model, a version of the Stanford Watershed Model. Incremental volumes of precipitation excess from many years of watershed simulation are compiled in a Runoff File. Thus the Runoff File contains precipitation excess generated from simulations which have accounted for soil properties and antecedent moisture conditions.

The physical characteristics of a watershed which influence the watershed response are input to UROS. The first step in preparing the input is segmenting the watershed into its elemental parts. Segments may be land surfaces, streams, or impoundments. These are called source area segments, channel segments, and storage segments, respectively. By defining which area drains to which stream and which stream drains to which impoundment, the linkage of the watershed is determined. The ordered array of segments and linkages is referred to as a watershed configuration.

For any given watershed configuration, UROS produces a hydrograph for each watershed segment for each sequence of rainfall excess in the Runoff File by routing the rainfall excess through the watershed. The largest peak flow from these hydrographs is determined for each water year and an annual peak flow series is compiled. Analysis of this series produces estimates of expected peak flow rates for specific return periods.

Despite the capability of UROS, it has not been widely used by practicing engineers. To some extent, this is due to the reluctance on the part of some engineers to become involved with computers. Additionally, the input requirements for UROS and the time requirements for preparing the input do not make it appealing for use on small watersheds when other methods are easier to use and when a high degree of accuracy in flow estimates is not required. Thus it is desirable to develop a simplified method that (1) can be used on small watersheds and (2) can closely approximate the results that would be obtained from simulations using UROS.

The purpose of this research is to develop a simple method of determining the effects of urbanization on expected flood peaks in small watersheds. The method is based on information obtained from computer simulation of a hypothetical watershed whose characteristics have been determined by analyzing actual watersheds in DeKalb County.

CHAPTER II

PROCEDURE

Three basic steps were followed in this research to develop a simple method for determining the effects of urbanization. The first step was to define a hypothetical watershed whose characteristics were representative of DeKalb County morphology. The physiography of the County was analyzed to determine representative values of stream density, hydraulic length of watersheds, and channel slope.

The second step was to systematically generate peak flow values for the hypothetical watershed through computer simulation. By changing the physical descriptions of the segments of the hypothetical watershed, different configurations of channel improvement and land surface imperviousness were represented. Each configuration produced peak flow values which could be associated with the physical description of the hypothetical watershed.

With these data generated, the third step in the investigation was to correlate the simulated peak flow values with the physical descriptors of the watershed. These descriptors, which included drainage area, percent imperviousness, road density, and percent of channels hydraulically modified, became the independent variables for a regression analysis. The mean and the standard deviation of the annual peak flow series, generated for each configuration of the hypothetical watershed, were the dependent variables.

By analyzing the final regression equations, a simple method for determining the effects of urbanization on peak flows from small watersheds was developed. The method assumed the effects of imperviousness and the effects of increased drainage efficiency to be separate. By so doing, the combined effects of these two watershed modifications were determined.

The Hypothetical Watershed

DeKalb County Physiography and Land Use Patterns

To generate the peak flow data through simulation, it was necessary to construct a hypothetical watershed whose morphology and land use characteristics are representative of DeKalb County, Georgia. Echo Branch sub-basin and Peachtree Branch sub-basin, both in the North Fork of Peachtree Creek watershed, were selected for analysis. Since both have been investigated under DeKalb County's Pilot Sub-basin Drainage Program (PSDP), much information was readily available about these watersheds (DeKalb County, 1976; DeKalb County, 1977).

Echo Branch sub-basin is a 2.5 square mile watershed completely developed in single family residential properties. Peachtree Branch sub-basin is also a 2.5 square mile watershed currently undergoing residential development. Both were selected for the PSDP because they are representative of the drainage systems existing throughout DeKalb County. Data for these watersheds were taken from 1" = 200' topographic maps. These maps are based on 1967 aerial photos, but the land uses and drainage system have been updated for these two watersheds by using more current aerial photos and field inspections.

Three watershed characteristics were investigated. They were stream density, stream bed slope, and hydraulic length of watershed. Drainage density is the ratio of the length of all channels in a watershed to the drainage area. This definition includes both natural and artificial channels as well as perennial, intermittent and ephemeral streams. For natural conditions, a headwater swale was not classified as an ephemeral stream. By this definition natural watershed drainage densities of 5.3 and 4.2 miles per square mile were determined for Peachtree Branch and Echo Branch respectively.

An investigation of 24 values of stream bed slope, measured at the watershed outlet, and the associated drainage area revealed that the slope and area are related (Figure 1). A regression analysis of these data produced the relationship

$$S = 0.15(A)^{-0.409} \tag{1}$$

where S is the channel slope in ft./ft. and A is the drainage area in acres. The correlation coefficient for this equation is -0.898 which indicates a moderately strong correlation.

Because channel slopes may vary considerably near the ridge line, another investigation was made to determine the average channel slope for headwater swales. Nineteen samples of 5-acre headwater areas taken at random from Peachtree Branch indicated a slope of 0.06 ft./ft. whereas the regression analysis indicated a somewhat steeper slope of 0.08 ft./ft.

Hydraulic length is a watershed shape factor that influences the lag between rainfall and runoff. Measuring from the drainage area outlet to the ridge line, 24 values of hydraulic length and the associated area were taken from Peachtree Branch and Echo Branch and a regression analysis performed. The equation developed was:

$$L = 272(A)^{0.532} \tag{2}$$

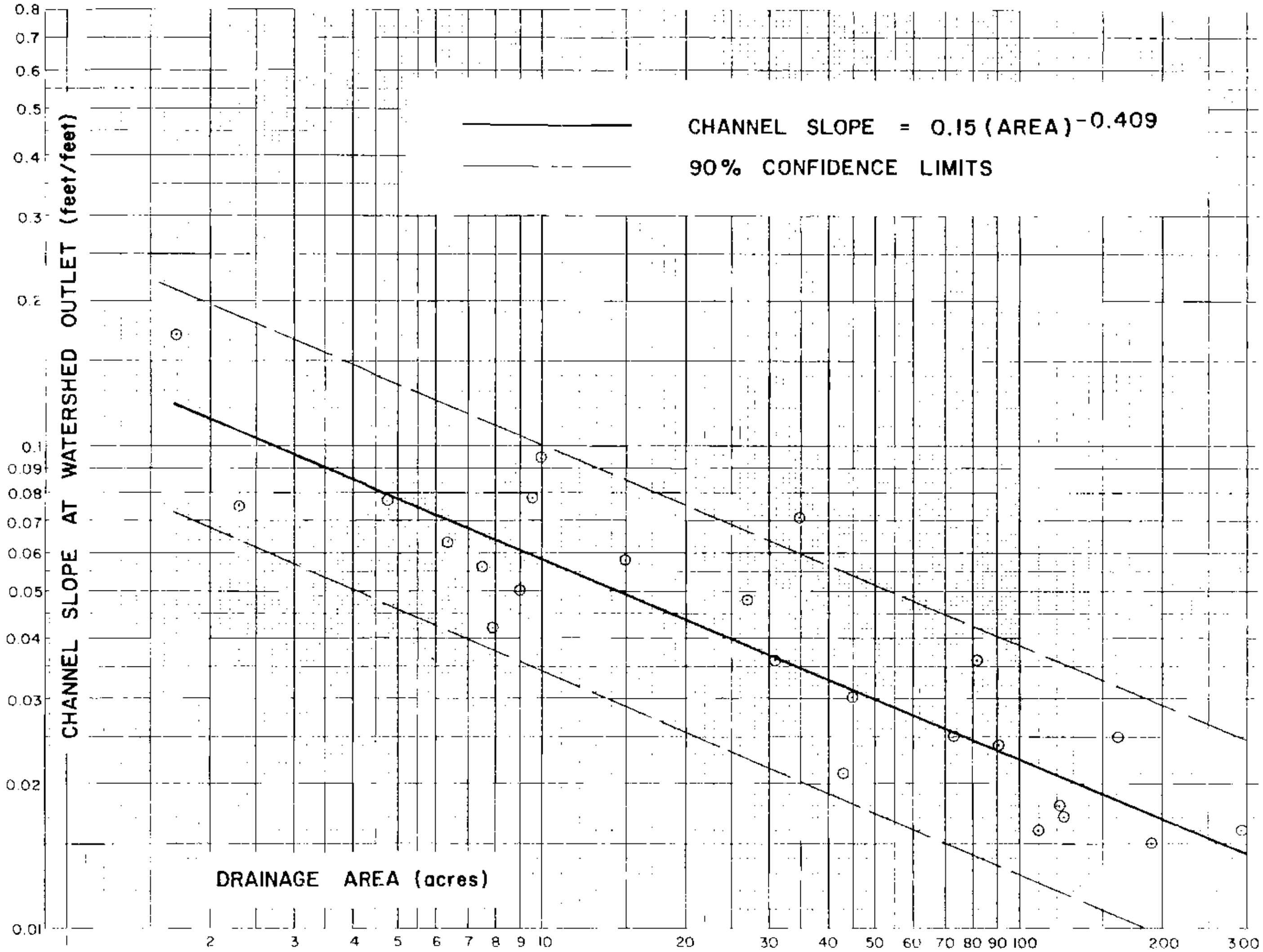

Figure 1. Channel Slope vs. Drainage Area.

where L is the hydraulic length of the watershed in feet and A is the area in acres. The correlation coefficient for this equation was 0.983 indicating a strong correlation. This equation can be compared to that published by the Soil Conservation Service (SCS, 1971). It should be noted that for the interval investigated, the SCS equation is within the 90 percent confidence limits of the DeKalb County equation (Figure 2).

Limitations of UROS

Before constructing the hypothetical watershed, it was necessary to analyze the structural limitations of the UROS computer model. Primary among these is the fact that UROS can only simulate 99 watershed segments in any one configuration; furthermore, no one configuration can accommodate more than either 50 area segments or 65 channel segments.

UROS uses the single linear reservoir (SLR) model to simulate runoff from unit source areas. This runoff can either be routed into the head of the next downstream channel or it may be collected laterally into a channel within the unit source area. Because of this flexibility, the opportunity exists to "double route" a hydrograph, once from the unit source area and again through the channel collecting the lateral inflow. This "double routing" produces a lower peak flow in the case of lateral inflow into the channel than from the unit source area discharging directly into the next downstream channel. A problem arises in comparative analysis if in one instance a channel is used within the source area and in another instance it is not. As the effect of the "double routing" is to mathematically reduce the peak flow, it introduces a bias into the model. To effectively deal with this situation, it is necessary to provide the same number of channels and source areas in both configurations.

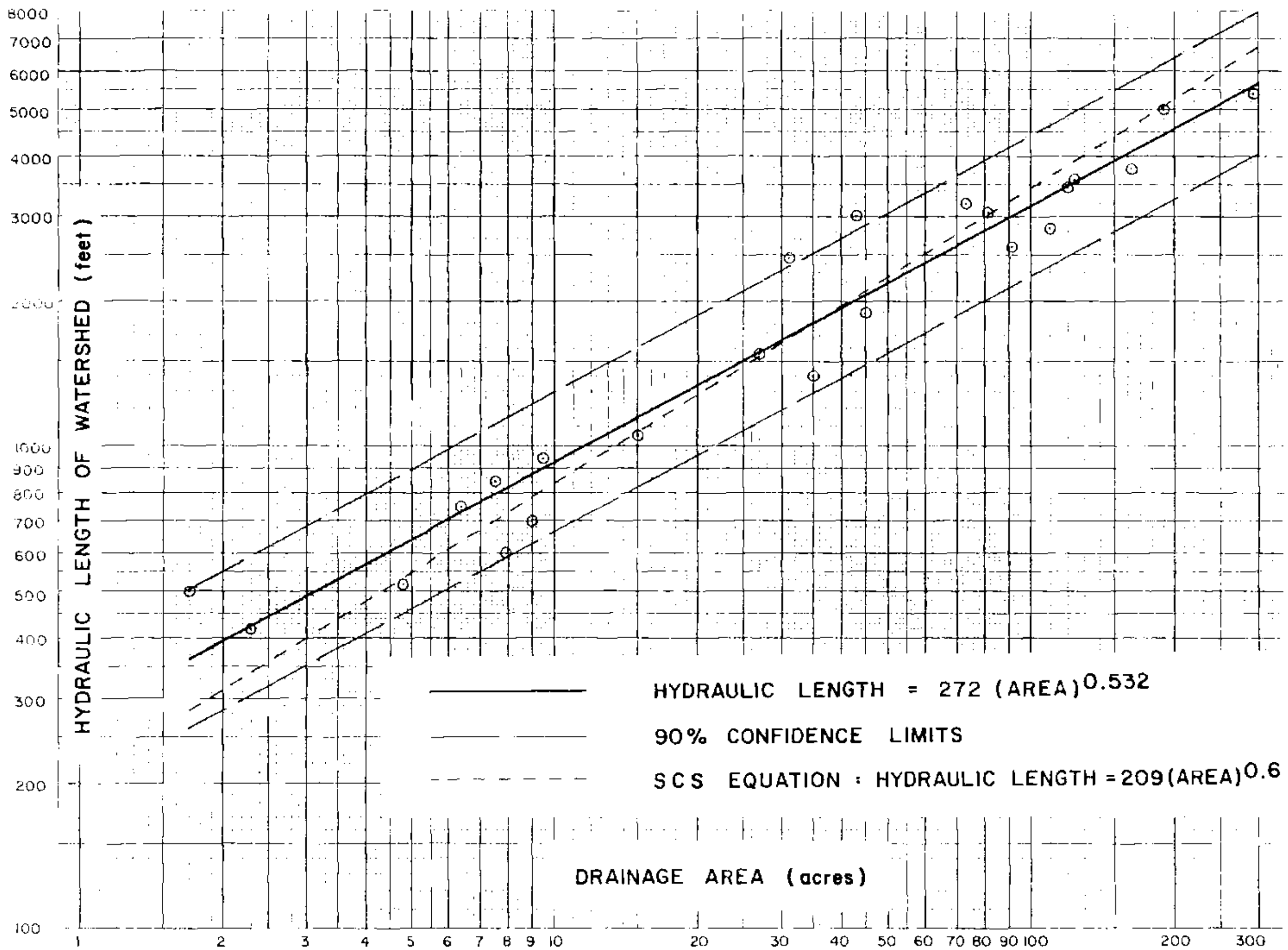

Figure 2. Hydraulic Length of Watershed vs. Drainage Area.

The channel as described may reflect either natural conditions or the effects of urbanization.

Hypothetical Watershed

The skeletal structure of the hypothetical watershed was a configuration of source area segments and channel segments. Because of the limitations of UROS, only 99 total segments could be simulated and each source area segment had to discharge laterally to a channel segment; therefore, the hypothetical watershed contained 48 source area segments, 48 channel segments, and 3 dummy segments which were used as calculating conveniences. Thirty-two of these channel segments were defined as "headwater" channel segments because they received runoff only from source area segments. The remaining 16 channel segments composed the "basic stream network" and each received lateral inflow from a source area segment and stream flow from one or more other channel segments.

The basic configuration of the hypothetical watershed had to be consistent with typical values of drainage density, hydraulic length of watersheds, and channel slope. Table 1 shows the drainage densities for the hypothetical watershed under natural conditions. Figure 3 shows the hydraulic length of watersheds at selected drainage areas in the hypothetical watershed plotted with the curve generated from the sample data for DeKalb County. This comparison verified the selected values of two watershed characteristics. Selection of channel lengths necessary to establish the values given for these characteristics is discussed later. Other characteristics of the source area and channel segments were selected to reflect various land uses and various degrees of drainage modification.

One of the concepts basic to this investigation is that roadway gutters act as extensions of the first order stream network. The

Table 1. Drainage Densities for the Hypothetical Watershed in Natural Conditions

Drainage Area (Acres)	Drainage Density (Miles per sq. mile)
5	0
13	4.66
26	4.66
39	4.66
49	3.71
101	4.20
208	4.66

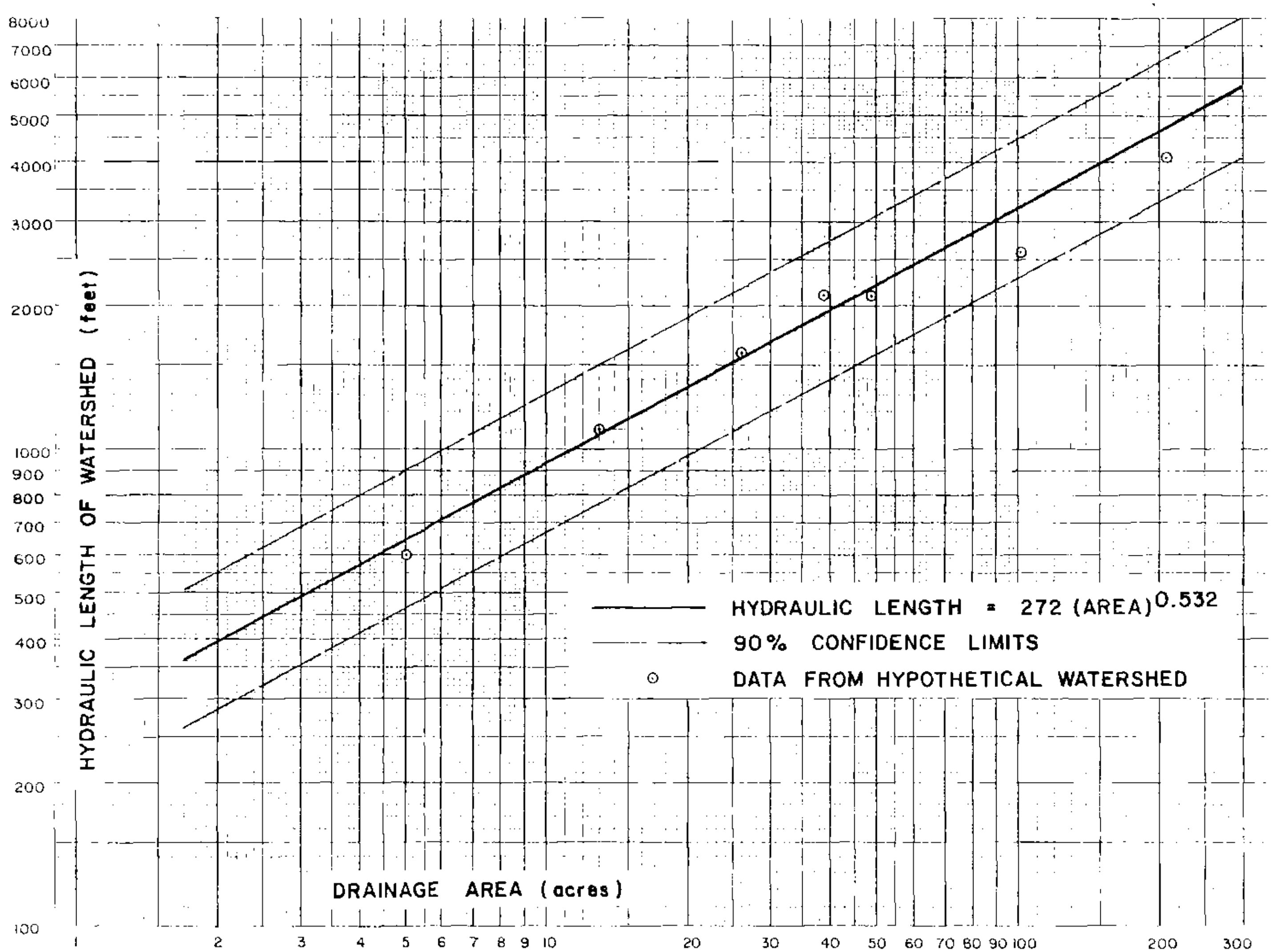

Figure 3. Hydraulic Length of Hypothetical Watershed Compared with Local Data.

introduction of such hydraulically efficient channels increases the drainage density and provides for a well drained watershed (Table 2). This reduces the flow times through the headwater areas and produces higher peak flows downstream. The physical description of a roadway gutter section was taken from the DeKalb County Standards (Figure 4). Each "gutter," as simulated, represented the hydraulic capacity of a pair of roadway gutters. Manning's "n" for all gutters was assigned a value of 0.015. Slopes of all gutters were assigned the value 0.05 ft./ft.

In a natural headwater area, the primary collector of overland flow was identified as a swale. This swale, shallow and highly rough, was considered hydraulically inefficient, and was not included as a first order stream when calculating drainage density. Accordingly, natural watersheds have a smaller stream density than do watersheds with roadway gutters (Tables 1 and 2). A parabolic geometry given by the equation:

$$Y = 0.05 \left[\frac{T}{2}\right]^2 \qquad (3)$$

was used to describe the swale cross-section (Figure 5). T is the top width of the swale and Y is the depth. The slope of the swale was set as 0.06 ft./ft. Manning's "n" was assumed to be 0.20. This estimate was based on results from studies by the Soil Conservation Service (SCS, 1969).

The headwater channel segment was defined as the smallest unit of the hypothetical watershed's channel network and was described as either a swale or a gutter. Each headwater channel segment was 600 feet long and collected runoff laterally from a 5 acre unit source area. Such an arrangement in a typical developed watershed contains 8-12 houses fronting on 600 feet of roadway (Figure 6).

Table 2. Drainage Densities for the Hypothetical Watershed With Gutters

Drainage Area (Acres)	Drainage Density (Miles per sq. mile)
5	14.55
13	15.85
26	15.85
39	15.85
49	15.58
101	15.72
208	15.85

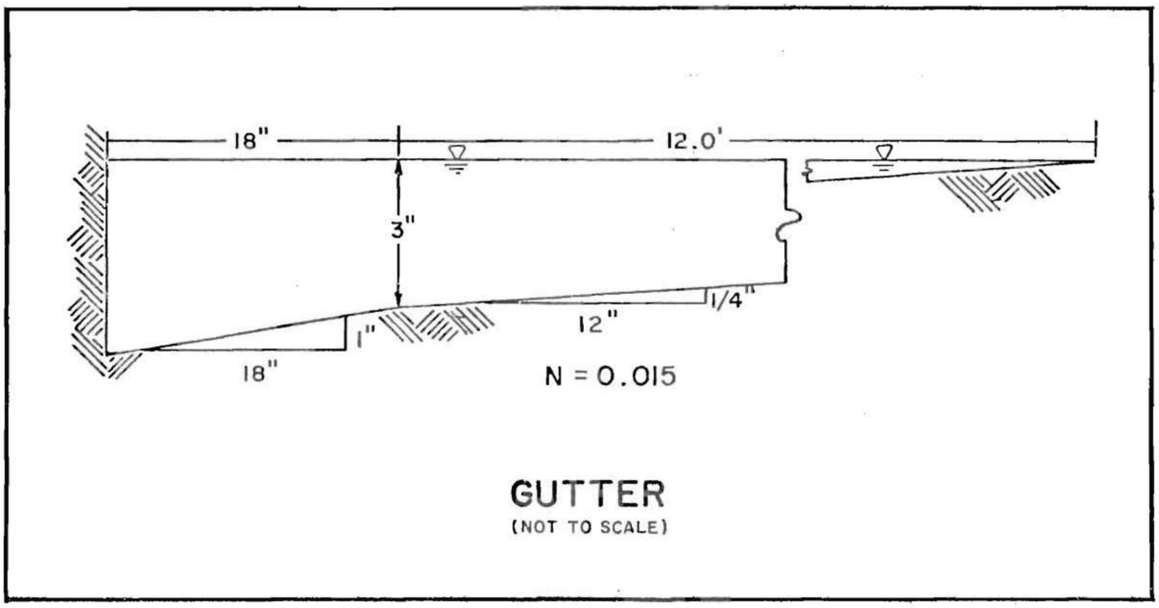

Figure 4. Standard Gutter Section Used in Hypothetical Watershed.

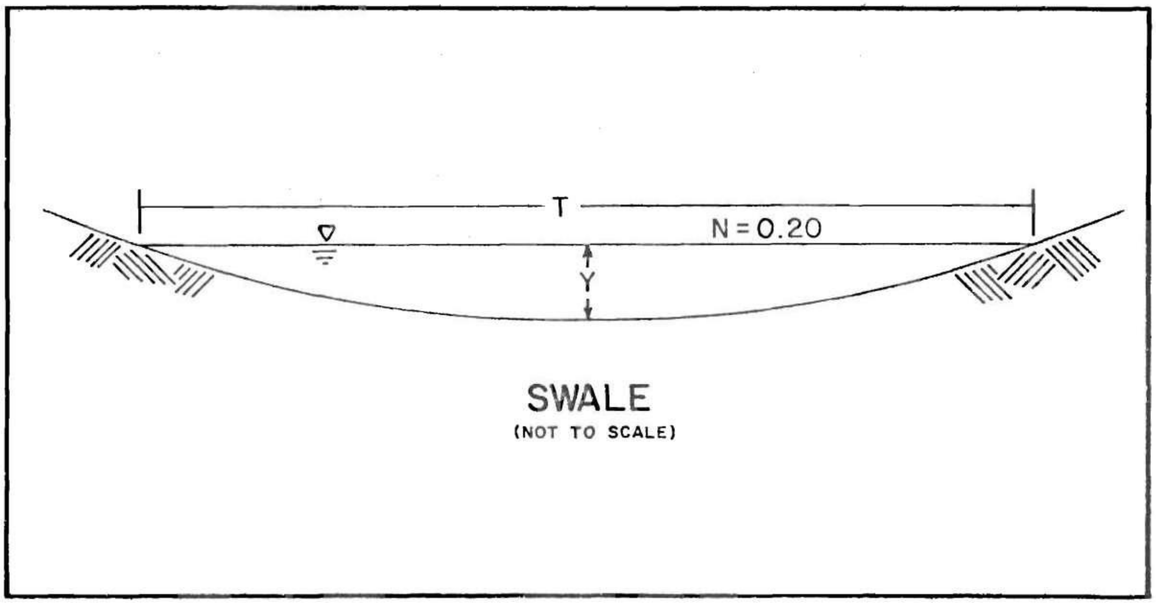

Figure 5. Cross-section of Natural Swale Used in Hypothetical Watershed.

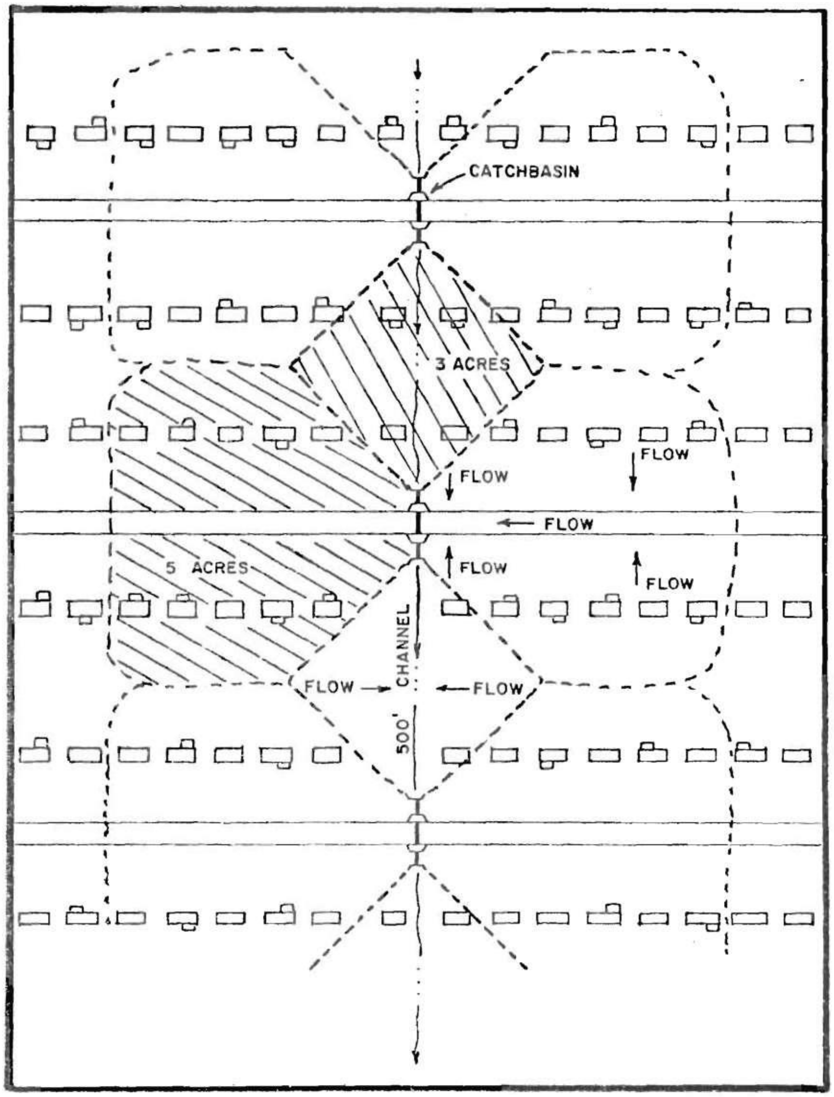

Figure 6. Typical Developed Unit Source Areas Used in Hypothetical Watershed.

Natural channel characteristics were selected based on the fact that bankfull conditions carry approximately the mean annual flow (Leopold, 1968). The channel slopes were calculated from Equation 1. The channel was assumed to be trapezoidal with 1H:2V side slopes and a base width to depth ratio of 2.0. The floodplains slope away from the banks at 10H:1V. Manning's "n" for the channel was assigned a value of 0.06 and for the floodplain the value was 0.10 (Figure 7). Because flows increase with drainage area, downstream channels must carry more flow at bankfull conditions; therefore, each channel segment has a different capacity. The discharge-cross-sectional area relationships for all natural channels including swales are shown in Figures 8-12.

As watersheds develop, channels are often improved or modified to reduce erosion and increase discharge capacity. It was assumed that in modified channels the channel slope and base width of the natural channel were retained and that the side slopes were laid back to 1H:1V to accommodate mortared rip-rapping for streambank stabilization (Figure 13). This laying back of the side slopes increases the bankfull cross-sectional area and the mortared rip-rap reduces the channel friction. These combined effects produce a modified channel whose bankfull capacity is approximately 3.5 times that of the natural channel. The discharge-cross-sectional area relationships for all modified channels including gutters are shown in Figures 8-12.

The distance along a reach of the basic stream network between confluences of headwater channel segments represents the distance between roadway crossings of the stream (Figure 14). Based on the sample watersheds, a unit length of 500 feet was selected for the basic stream network

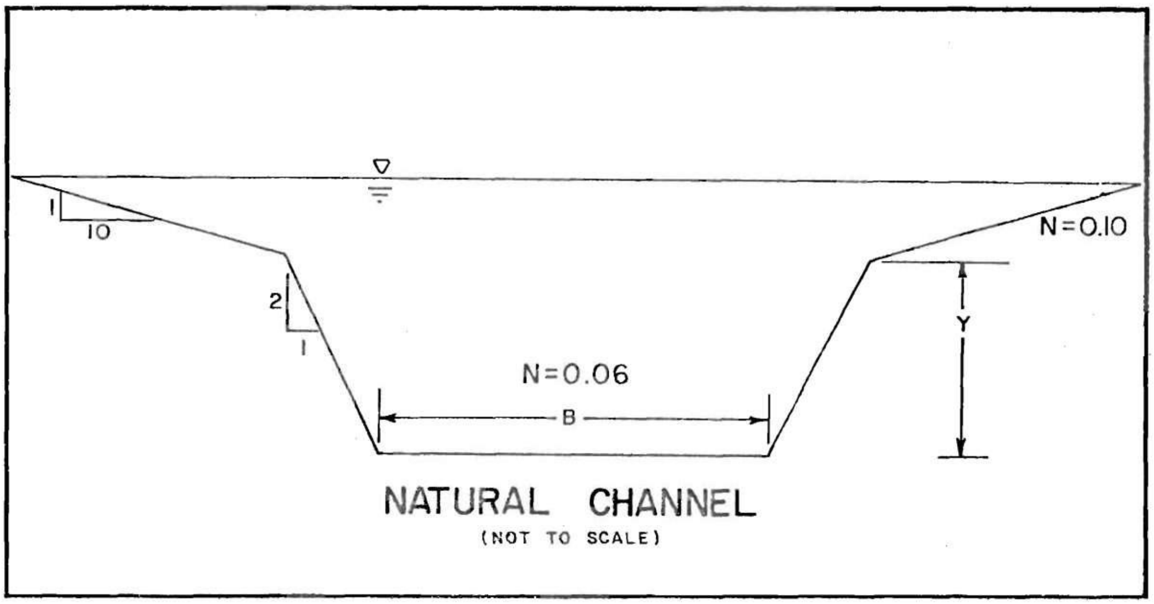

Figure 7. Cross-section of Natural Channel Used in Hypothetical Watershed.

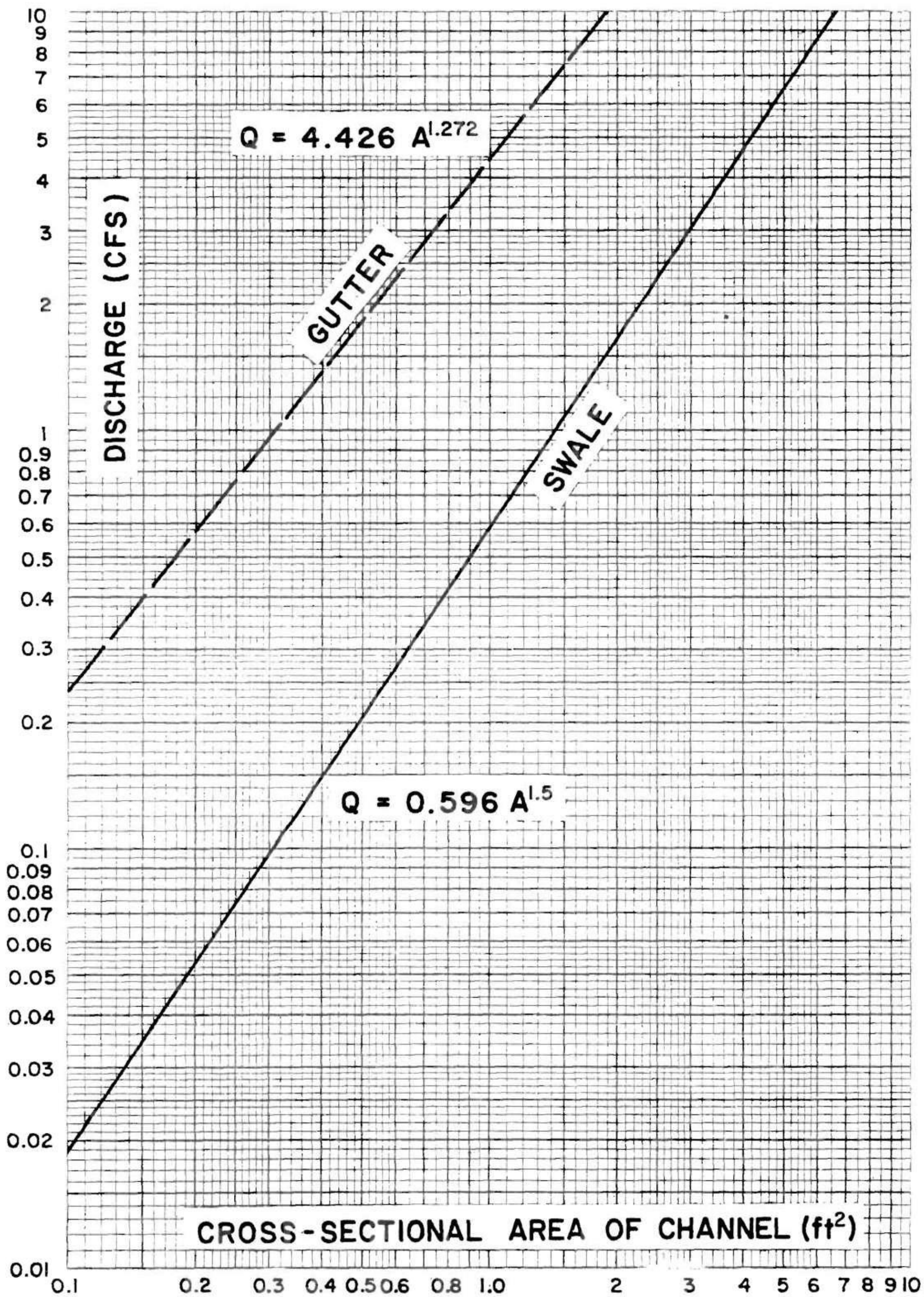

Figure 8. Discharge - Cross-sectional Area Relationships for Swales and Gutters.

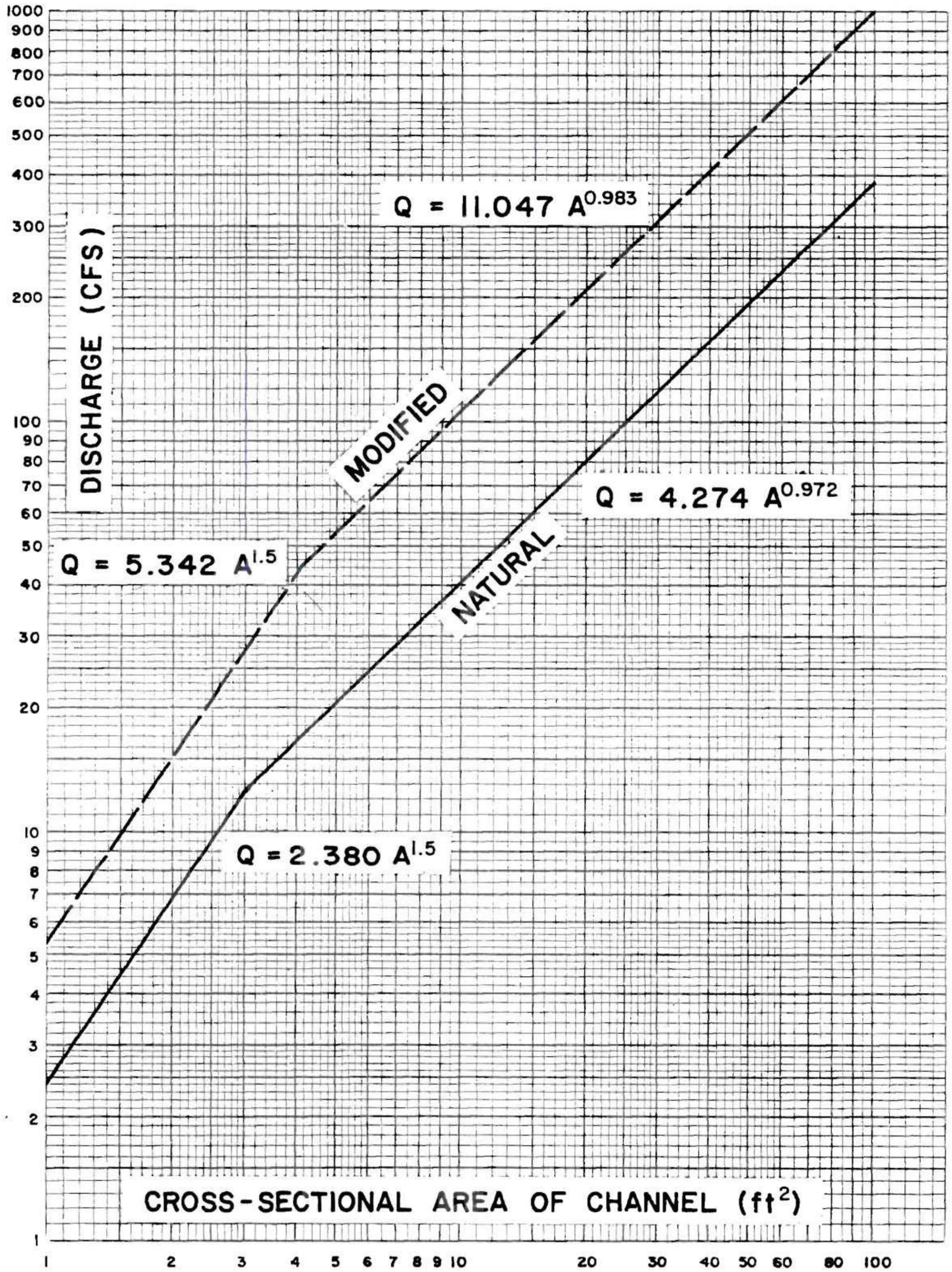

Figure 9. Discharge - Cross-sectional Area Relationships for Natural and Modified Channel Segments #57, 69, 81, and 92.

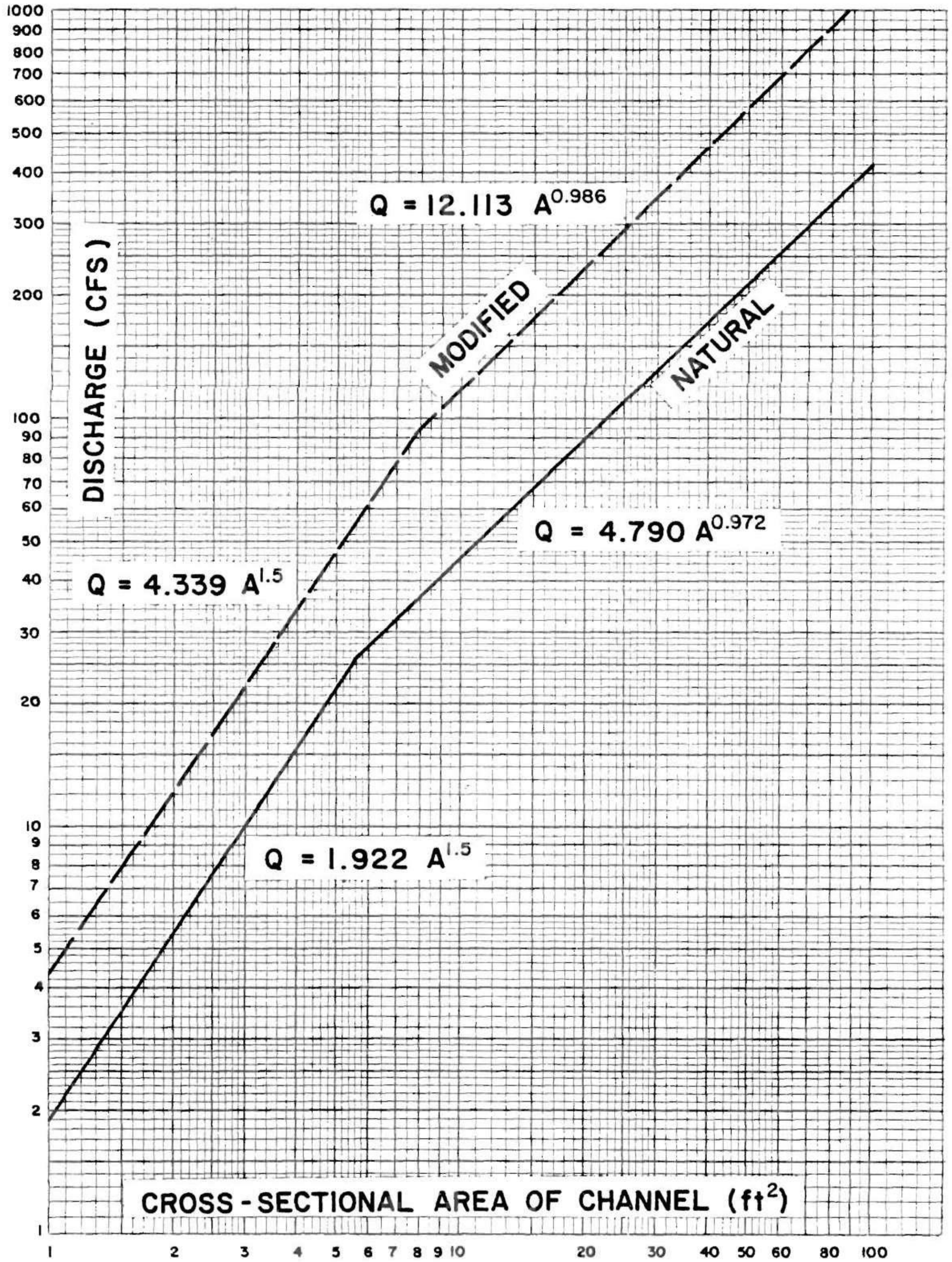

Figure 10. Discharge - Cross-sectional Area Relationships for Natural and Modified Channel Segments #58, 70, 82, and 93.

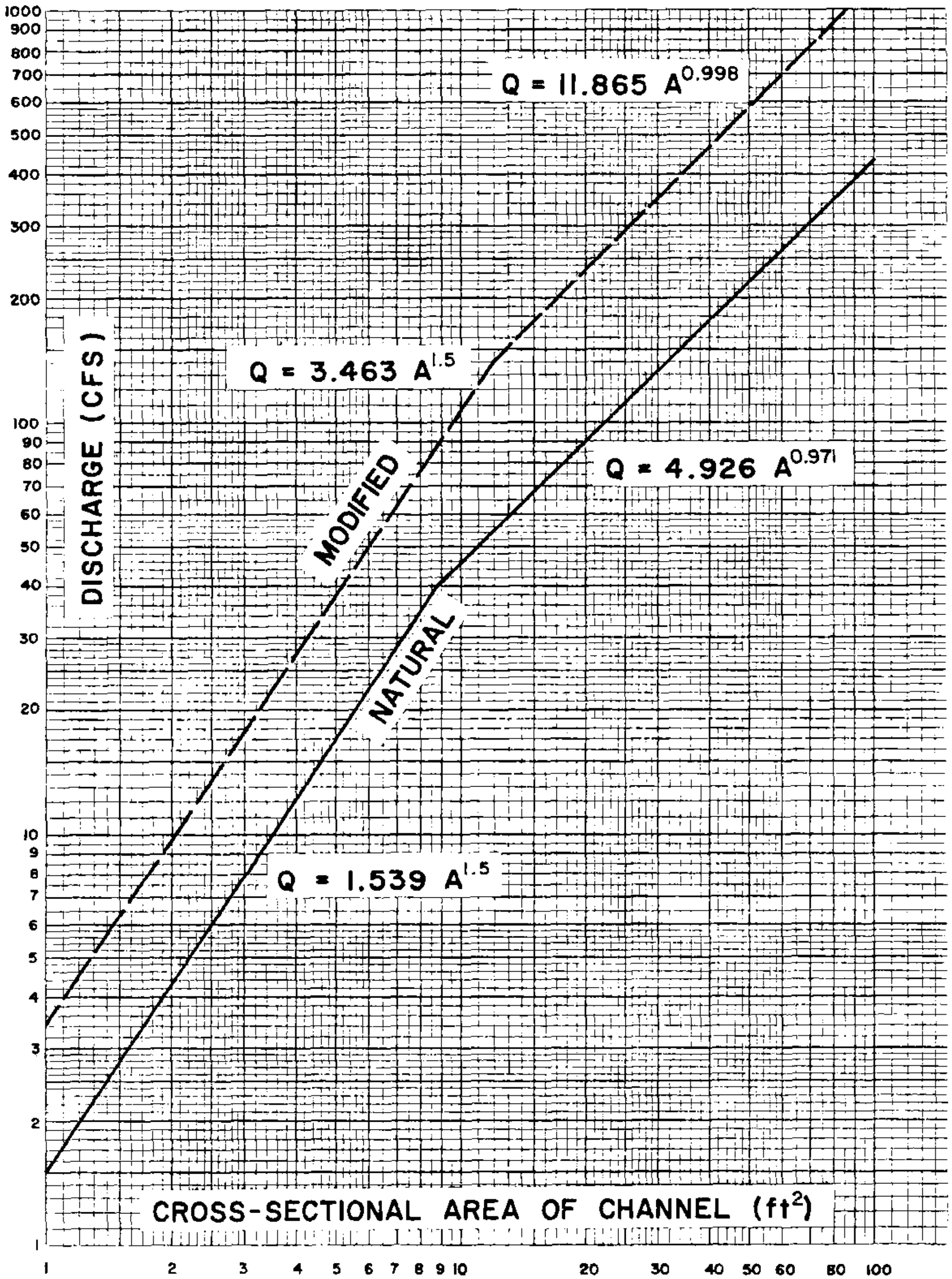

Figure 11. Discharge - Cross-sectional Area Relationships for Natural and Modified Channel Segments #59, 71, 83, and 94.

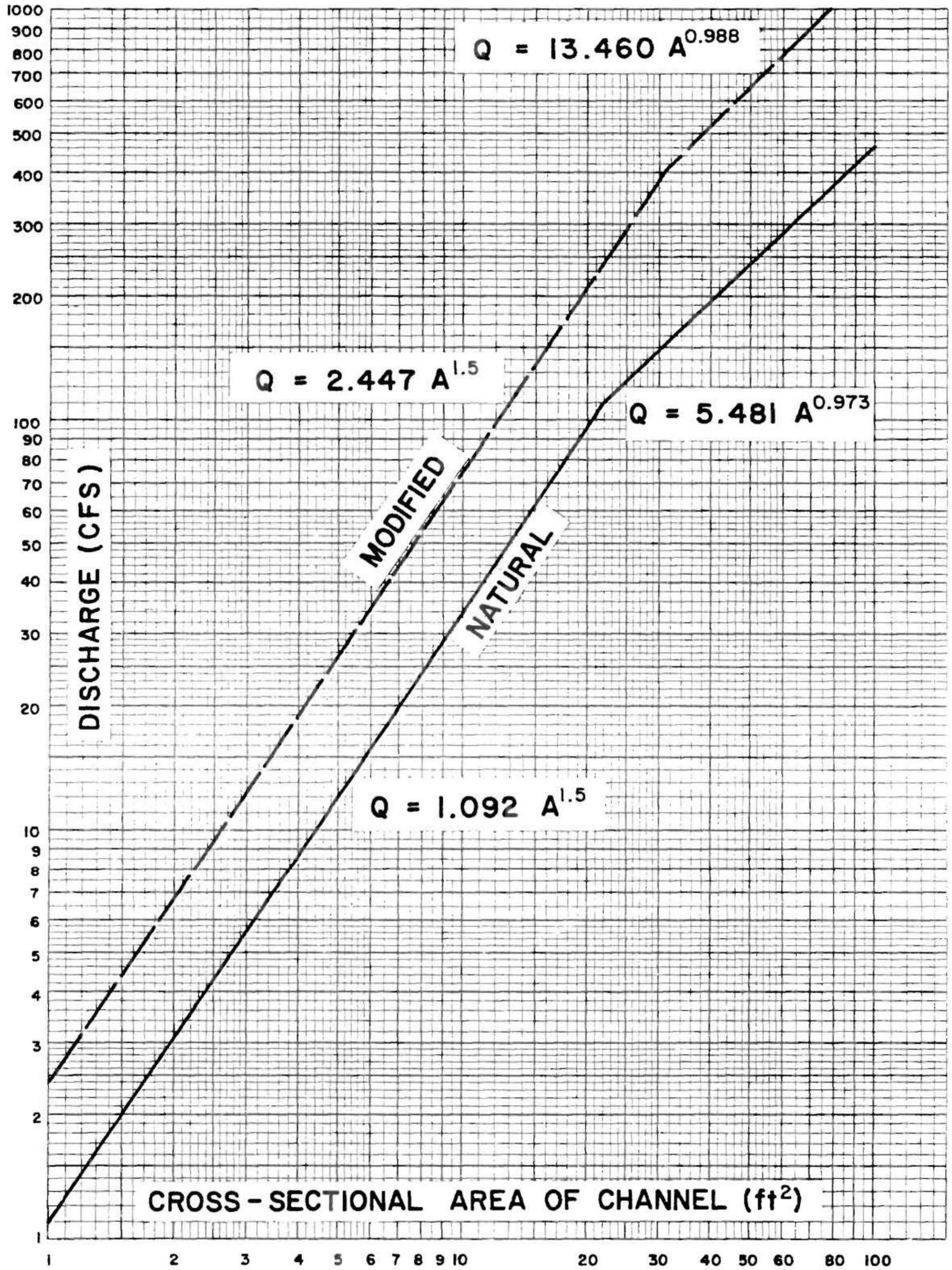

Figure 12. Discharge - Cross-sectional Area Relationships for Natural and Modified Channel Segments #95, 96, 97, and 98.

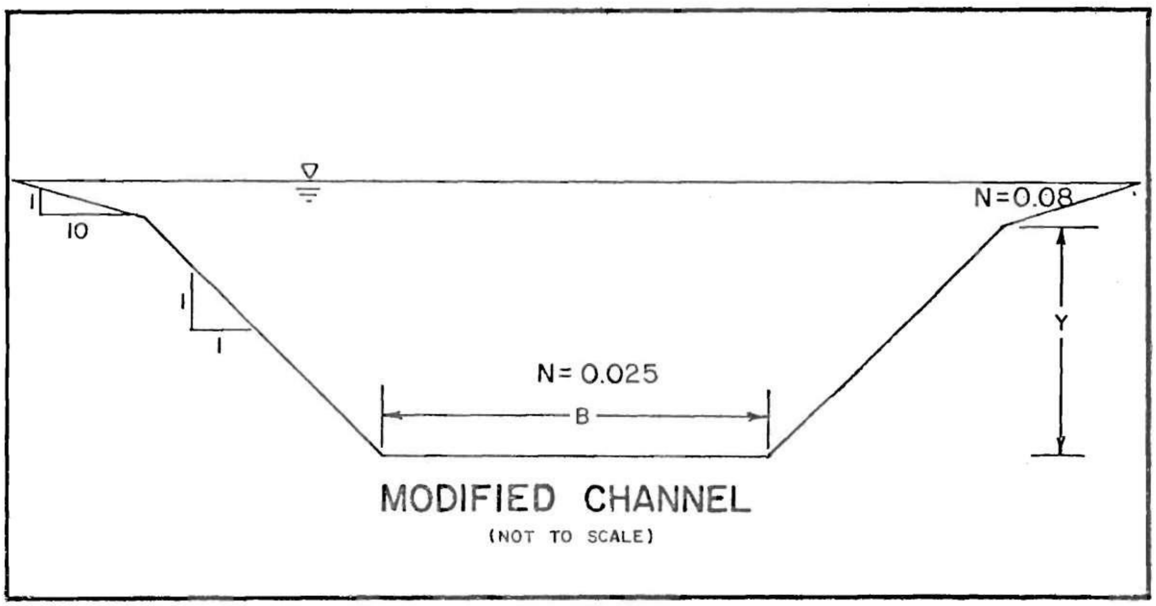

Figure 13. Cross-section of Modified Channel Used in Hypothetical Watershed.

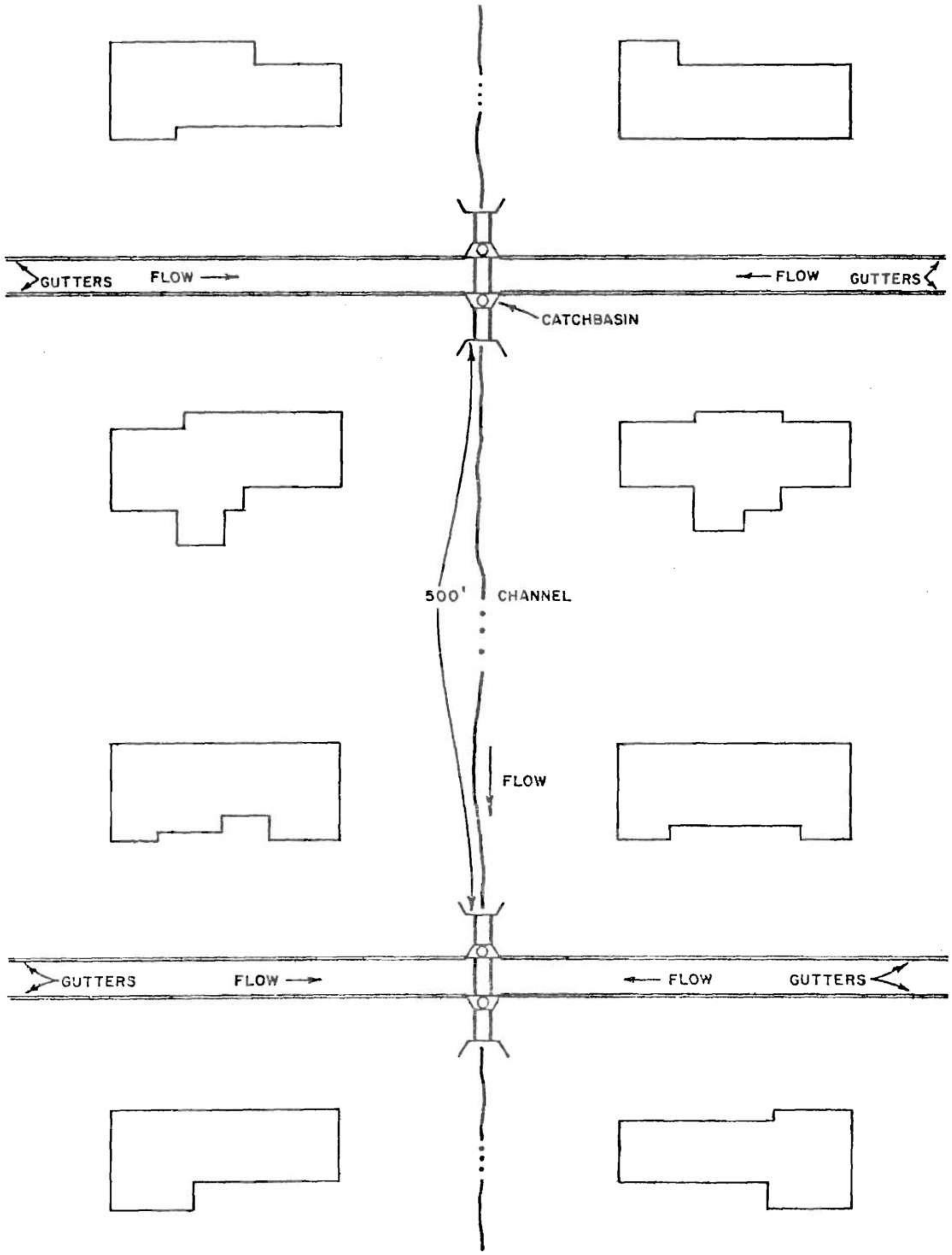

Figure 14. Representation of Roadway Crossings of a Stream.

channel segments. Each of the 16 channel segments composing the basic stream network collects runoff not only from its tributary channels but also from a local adjacent land surface; therefore, a 3-acre unit source area was designated to contribute laterally to each 500-foot unit channel segment.

The 208-acre hypothetical watershed contains four symmetric sub-basins. Each sub-basin contains eight 5-acre headwater source area segments, eight 600-foot headwater channel segments, three 500-foot basic stream network channel segments, and three 3-acre local inflow source area segments. Two of these 49-acre sub-basins contribute at their confluence to a 2000-foot reach of the basic stream network. This reach is composed of four 500-foot channel segments each of which receives local lateral inflow from a 3-acre source area segment. The downstream end of the 2000-ft. reach coincides with the confluence of the other two 49-acre sub-basins; therefore, the total area of the hypothetical watershed is 208 acres or approximately one-third of a square mile. Figure 15 shows the hypothetical watershed schematically represented for modelling with UROS.

Systematic Generation of Peak Flow Data

Selection and Combination of Parameters

The various possible combinations of swales and gutters, natural and modified channels, and pervious and impervious areas presents a large number of watershed configurations. By careful selection of possible combinations, this investigation was limited to consideration of 64 unique configurations.

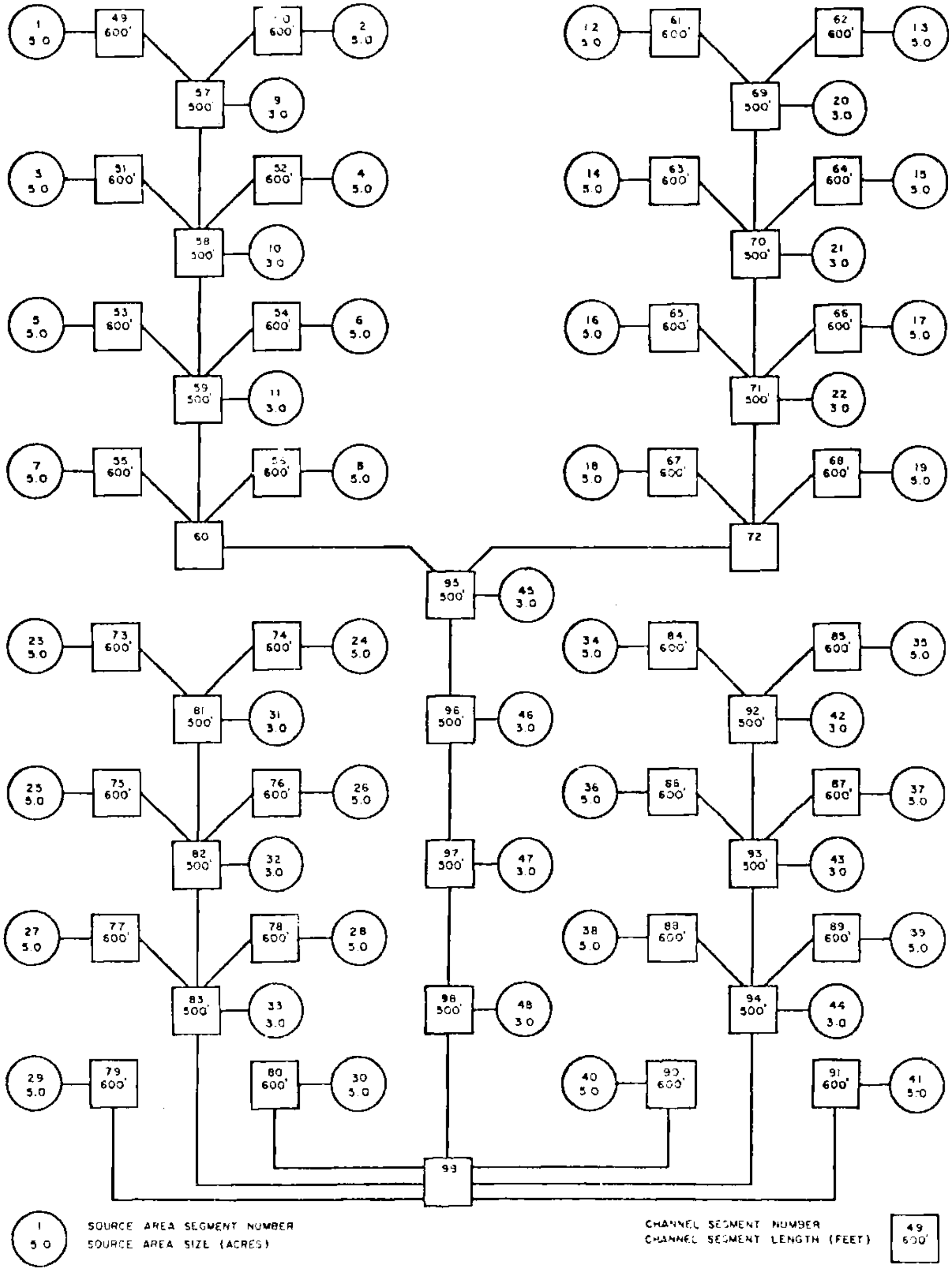

Figure 15. Schematic of the Hypothetical Watershed.

The hydraulic efficiency of a headwater channel segment depended on whether it was designated swale or gutter. Road density was selected as the appropriate measure of this hydraulic efficiency. The road density is the ratio of the total length of headwater channel segments designated as gutters to the drainage area under consideration. For the 5-acre headwater source area segment and its 600-foot headwater channel segment, road densities were either 0 or 14.55 miles per square mile. For the complete 208-acre hypothetical watershed, values from 0 to 11.19 miles per square mile were possible.

With the large number of possible combinations of swales and gutters, it was necessary to select certain combinations for simulation. Because the hypothetical watershed contained 4 symmetric sub-basins, swale-gutter combinations were arranged identically in each sub-basin. Swales and gutters were arranged in pairs according to their confluences with the basic stream network. Starting with a watershed containing only swales, gutters were introduced into each sub-basin by converting the pair of swales nearest the ridge and then proceeding downstream. Four combinations of swales and gutters were selected for simulation (Figure 16).

The hydraulic characteristics of the basic stream network may be those of a natural stream system, a partially modified system, or a fully modified system. Each of the 16 basic stream network channel segments was assigned the properties of a natural channel or a modified channel. The percent of the basic stream network channel length which was hydraulically modified was selected as the appropriate measure of the degree of modification. As with road density the actual value of the parameter, percent of channel hydraulically modified, was dependent on the number of modified

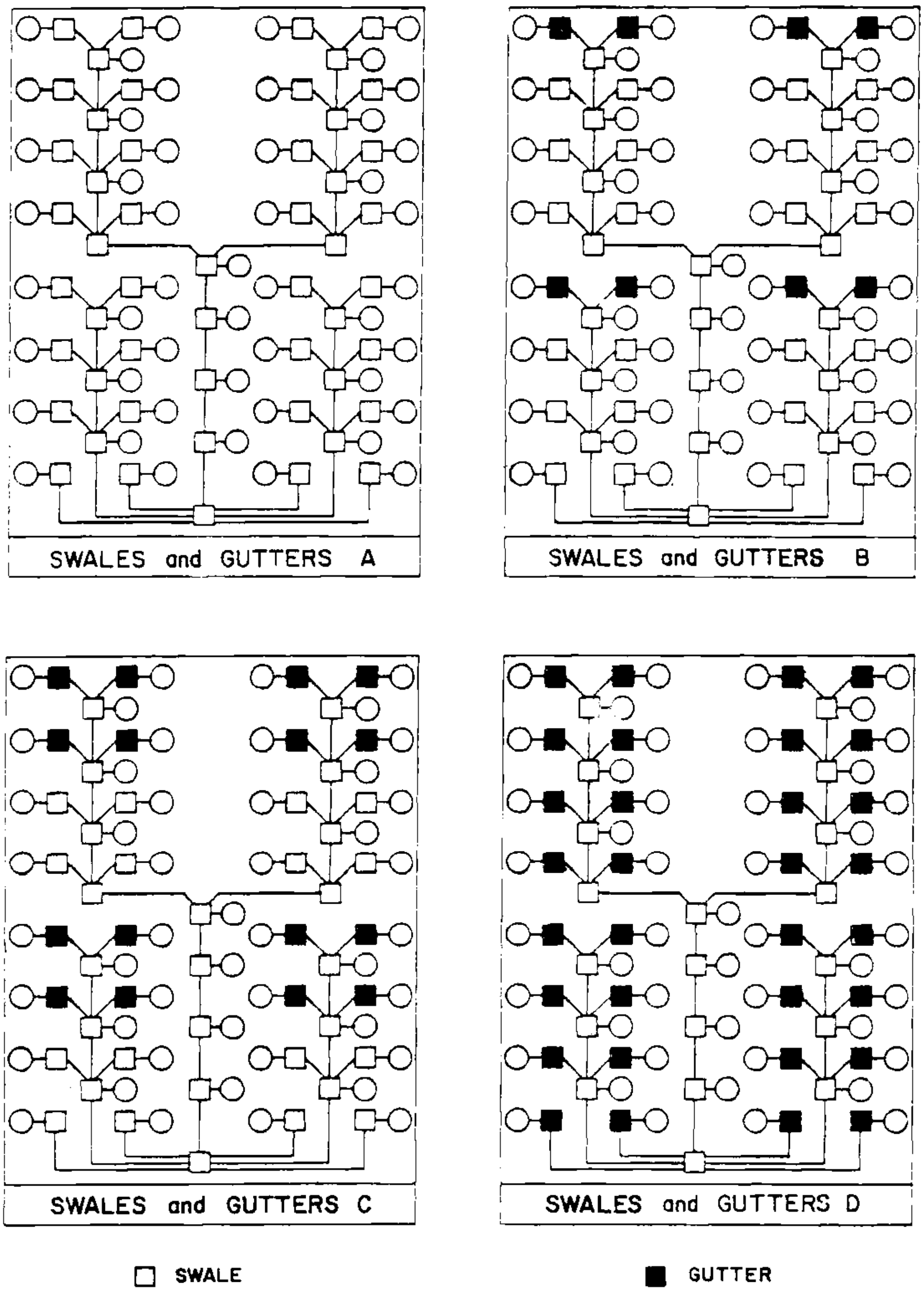

Figure 16. Four Combinations of Swales and Gutters.

basic stream network channel segments within the area under consideration. For the smallest configuration containing a basic stream network channel segment, the parameter value was either 0 percent or 100 percent. For the full 208-acre watershed, parameter values in increments of 6. 5 percent from 0 percent to 100 percent were possible.

Four combinations of natural channels and modified channels were selected. It was noted that in the sample watersheds the most upstream reaches of the channels were the first to be modified. Accordingly, starting with a watershed containing only natural channels, the basic stream network channel segments nearest the ridge line were the first to be converted to modified channels. This was done symmetrically in the four identical sub-basins. The four selected combinations of natural channels and modified channels are shown in Figure 17.

Drainage area is the primary indicator of expected peak flows and must be included as a parameter related to expected peak flows. Because the hypothetical watershed had four symmetric sub-basins and because both head-water channel segments and basic stream network channel segments were arranged consistently in all four sub-basins, there were only seven possible unique drainage area values. These were 5, 13, 26, 39, 49, 101, and 208 acres (Figure 18).

The percent of land surface that is impervious was selected as a fourth variable to be used in simulation. To avoid the confounding effect of having different values of imperviousness in different parts of the hypothetical watershed for a single configuration, the percent of imperviousness was defined to be uniform for all source area segments in any one configuration. Values of 0, 20, 40, and 100 percent were selected

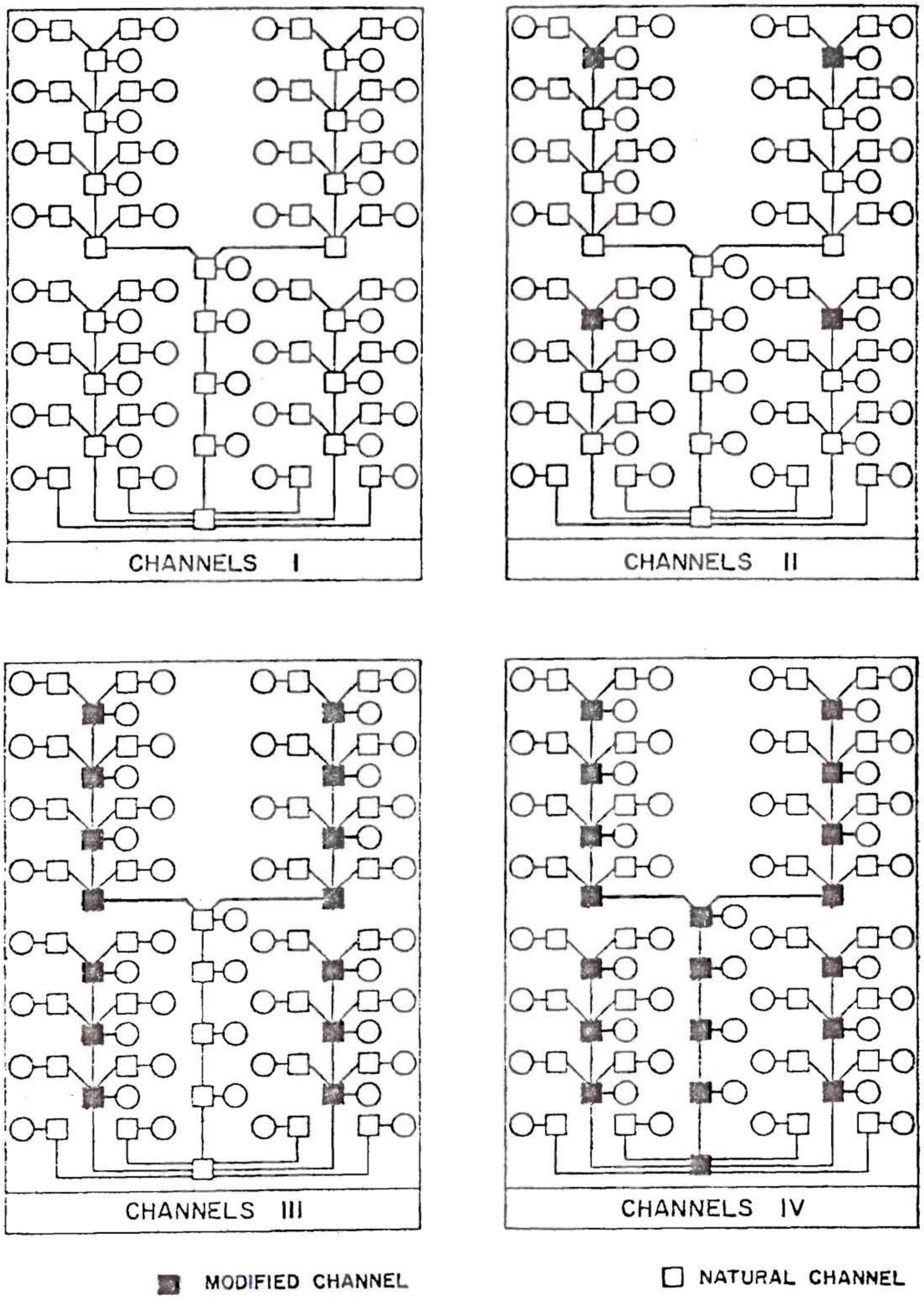

Figure 17. Four Combinations of Natural Channels and Modified Channels.

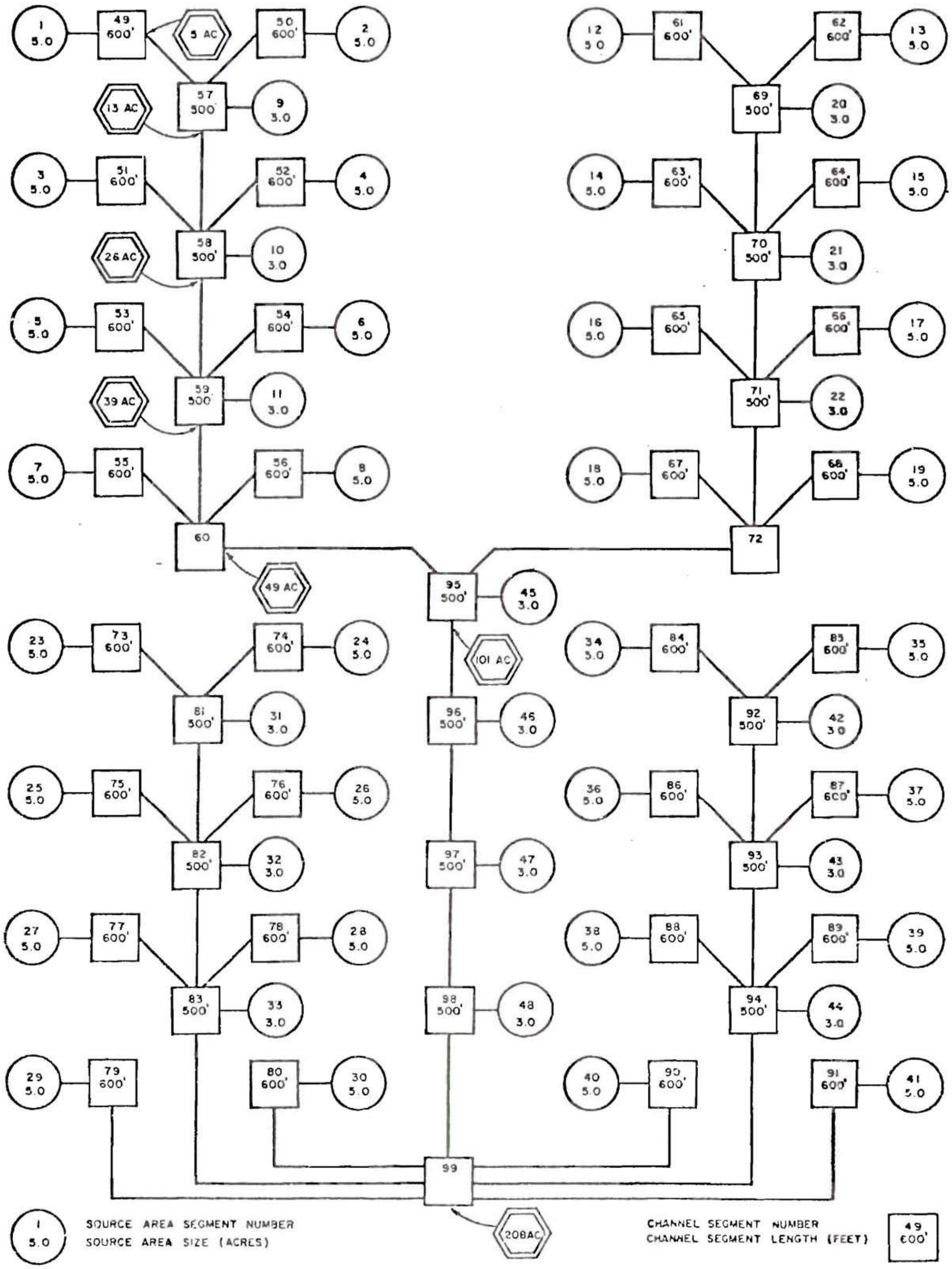

Figure 18. Location of Selected Drainage Area Outlets in Schematic of the Hypothetical Watershed.

for the percent impervious parameter.

The exceedence probability, or return period, was also selected as a parameter to be considered. The UROS computer model analyzes an annual peak flow series and produces a mean and standard deviation for this series. With values for these two dependent variables, an extreme value Type 1 or Gumbel probability density function was used to produce estimates of expected peak flow values for specific return periods.

The five parameters which are related to expected peak flows from urban watersheds are drainage area, road density, percent of channels hydraulically modified, percent imperviousness, and return period. Because each watershed configuration contains all drainage area sizes and because each UROS simulation produces a mean and a standard deviation of the annual peak flow series, only three parameters had to be combined into unique configurations. With 4 combinations of swales and gutters, 4 combinations of natural channels and modified channels, and 4 values of percent imperviousness, 64 final configurations of the hypothetical watershed were prepared for simulation with UROS (Table 3).

Simulation with UROS

Each of the 64 watershed configurations was modelled with UROS. The hydrologic response of each configuration to the historical pattern of precipitation excess, available from a previous analysis and stored in a Runoff File, was simulated. From these simulations, using the 1-minute Runoff Files for Soil 2 and Soil 4, the mean and standard deviation of the annual peak flow series were established and recorded along with the values of parameters drainage area, percent impervious, road density, and percent of channels hydraulically modified. This investigation produced 284 unique sets of parameter values and simulated flows (Appendix A).

Table 3. Sixty-four Configurations of the Hypothetical Watershed

	Combination of Swales and Gutters (See Figure 16)	Combination of Natural and Modified Channels (See Figure 17)
	A	I
All 16 Combinations of	A	II
Swales, Gutters,	A	III
Natural Channels, and	A	IV
Modified Channels	B	I
were combined with	B	II
each of four values of	B	III
percent imperviousness:	B	IV
0%, 20%, 40%, 100%.	C	I
	C	II
	C	III
	C	IV
	D	I
	D	II
	D	III
	D	IV

Correlation of Peak Flow Values with Simulation Parameters

It was assumed that the log of the peak flow value could be correlated with the log of the watershed descriptor values. The basic relationship selected was: $\log Q_T = f(\log A, \log I, \log RD, \log HM)$, where Q is the expected peak flow at return period T, A is the drainage area, I is the percent of area impervious, RD is the road density, and HM is the percent of channel length hydraulically modified.

The Statistical Package for the Social Sciences (SPSS) was selected to perform a multiple regression analysis on the 284 sets of simulated data. This analysis produced equations for the mean and standard deviation of the annual peak flow series. Five additional equations were produced for expected peak flow rates for specific return periods assuming a given probability density function. The residuals were analyzed and changes in the regression arguments were made as necessary.

CHAPTER III

RESULTS

Regression Equations

The Statistical Package for the Social Sciences (SPSS) was used to perform multiple regression analyses on the data produced from simulations of the hypothetical watershed with UROS. The regression arguments first selected for each parameter are given in Table 4. Regression with these arguments generated reasonable equations for the mean and standard deviation of the annual peak flow series, but a plot of the regression residuals revealed a systematic variation with respect to the percent imperviousness. A revised argument, $(\log(1+I))^n$ where I is the decimal fraction of impervious area and n is some exponent between 1.1 and 1.9, was selected to remove these variations. No pattern of regression residuals was noted for the other parameters. The final regression arguments are given in Table 5.

Seven equations, all with correlation coefficients of at least 0.999, were generated from the data and are given in Table 6. The first two equations, which produce the mean and standard deviation of an annual peak flow series, can be used to estimate the expected peak flow value for any return period using a selected probability density function. The other five equations are for estimating expected peak flow values for certain return periods assuming an extreme value Type 1 or Gumbel distribution.

The arguments were entered into the regression equation in the

Table 4. Regression Arguments First Selected for Analysis

Equation	Parameter			
Peak Flow	Area	Imperviousness	Road Density	Channels
log Q	log A	log (1+I)	log (1+RD)	log (1+HM)

Q = expected peak flow in cubic feet per second
A = drainage area in acres
I = decimal fraction of impervious area
RD = road density in miles per square mile
HM = decimal fraction of channel length hydraulically modified

Table 5. Final Regression Arguments Selected for Analysis

Equation	Parameter			
Peak Flow	Area	Imperviousness	Road Density	Channels
$\log \bar{Q}$	log A	$(\log(1+I))^{1.1}$	log (1+RD)	log (1+HM)
log SD	log A	$(\log(1+I))^{1.9}$	log (1+RD)	log (1+HM)
$\log Q_5$	log A	$(\log(1+I))^{1.2}$	log (1+RD)	log (1+HM)
$\log Q_{10}$	log A	$(\log(1+I))^{1.3}$	log (1+RD)	log (1+HM)
$\log Q_{25}$	log A	$(\log(1+I))^{1.4}$	log (1+RD)	log (1+HM)
$\log Q_{50}$	log A	$(\log(1+I))^{1.45}$	log (1+RD)	log (1+HM)
$\log Q_{100}$	log A	$(\log(1+I))^{1.45}$	log (1+RD)	log (1+HM)

$\bar{Q}$ = expected mean annual peak flow in CFS
SD = standard deviation of the annual peak flow series in CFS
Q_T = expected peak flow for return period T, in CFS
A = drainage area in acres
I = decimal fraction of impervious area
RD = road density in miles per square mile
HM = decimal fraction of channel length hydraulically modified

Table 6. Final Regression Equations for Estimating Expected Peak Flows

$$\overline{Q} = 1.319(A)^{0.949}\left[(1+I)^{1.657}\right]^{\left[\log(1+I)\right]^{0.1}}(1+RD)^{.044}(1+HM)^{0.116} \tag{4}$$

$$SD \quad 0.811(A)^{0.956}\left[(1+I)^{2.890}\right]^{\left[\log(1+I)\right]^{0.9}}(1+HM)^{0.124}(1+RD)^{0.020} \tag{5}$$

$$Q_5 = 1.895(A)^{0.951}\left[(1+I)^{1.704}\right]^{\left[\log(1+I)\right]^{0.2}}(1+RD)^{0.037}(1+HM)^{0.119} \tag{6}$$

$$Q_{10} = 2.381(A)^{0.952}\left[(1+I)^{1.824}\right]^{\left[\log(1+I)\right]^{0.3}}(1+HM)^{0.120}(1+RD)^{0.034} \tag{7}$$

$$Q_{25} = 2.992(A)^{0.952}\left[(1+I)^{1.964}\right]^{\left[\log(1+I)\right]^{0.4}}(1+HM)^{0.120}(1+RD)^{0.032} \tag{8}$$

$$Q_{50} = 3.441(A)^{0.953}\left[(1+I)^{2.035}\right]^{\left[\log(1+I)\right]^{0.45}}(1+HM)^{0.121}(1+RD)^{0.030} \tag{9}$$

$$Q_{100} = 3.864(A)^{.953}\left[(1+I)^{2.004}\right]^{\left[\log(1+I)\right]^{0.45}}(1+HM)^{0.121}(1+RD)^{0.029} \tag{10}$$

$\overline{Q}$ = expected mean annual peak flow in CFS
SD = standard deviation of the annual peak flow series in CFS
Q_T = expected peak flow for return period T, in CFS
A = drainage area in acres
I = decimal fraction of impervious area
RD = road density in miles per square mile
HM = decimal fraction of channel length hydraulically modified

order of the unexplained variance which each removed. The drainage area and percent imperviousness were responsible for explaining 99 percent of the variance in the equations. The road density and percent of channels hydraulically modified explained only a very small portion of the variance; nevertheless, it is interesting to note that for the mean annual and 5-year peak flow equations, the swales and gutters had a greater influence in removing the variance than did the channels. For the higher return period equations, and the standard deviation, the role was reversed with the influence of the channels dominating the influence of the swales and gutters.

Mean Annual Flow Estimates for Natural Watersheds

An estimate of expected mean annual peak flow for small natural watersheds was developed from the regression equations. Taking the equation for mean annual flow, Equation 4, and dropping the terms for urbanization yields:

$$\overline{Q} = 1.32\ A^{0.949} \tag{11}$$

where $\overline{Q}$ is the expected mean annual peak flow in cfs and A is the drainage area in acres. Equation 11 is plotted in Figure 19.

Equation 11 is based on simulations using 48 source area segments and 48 channel segments to model 208 acres. The effect of this extensive watershed segmentation was investigated. Three simulations were made using different source area sizes to investigate the impact of modelling a watershed with a few large source areas as opposed to many small source areas. The first simulation modelled selected watershed sizes as single source areas. No channel segments were used. The mean annual peak flows

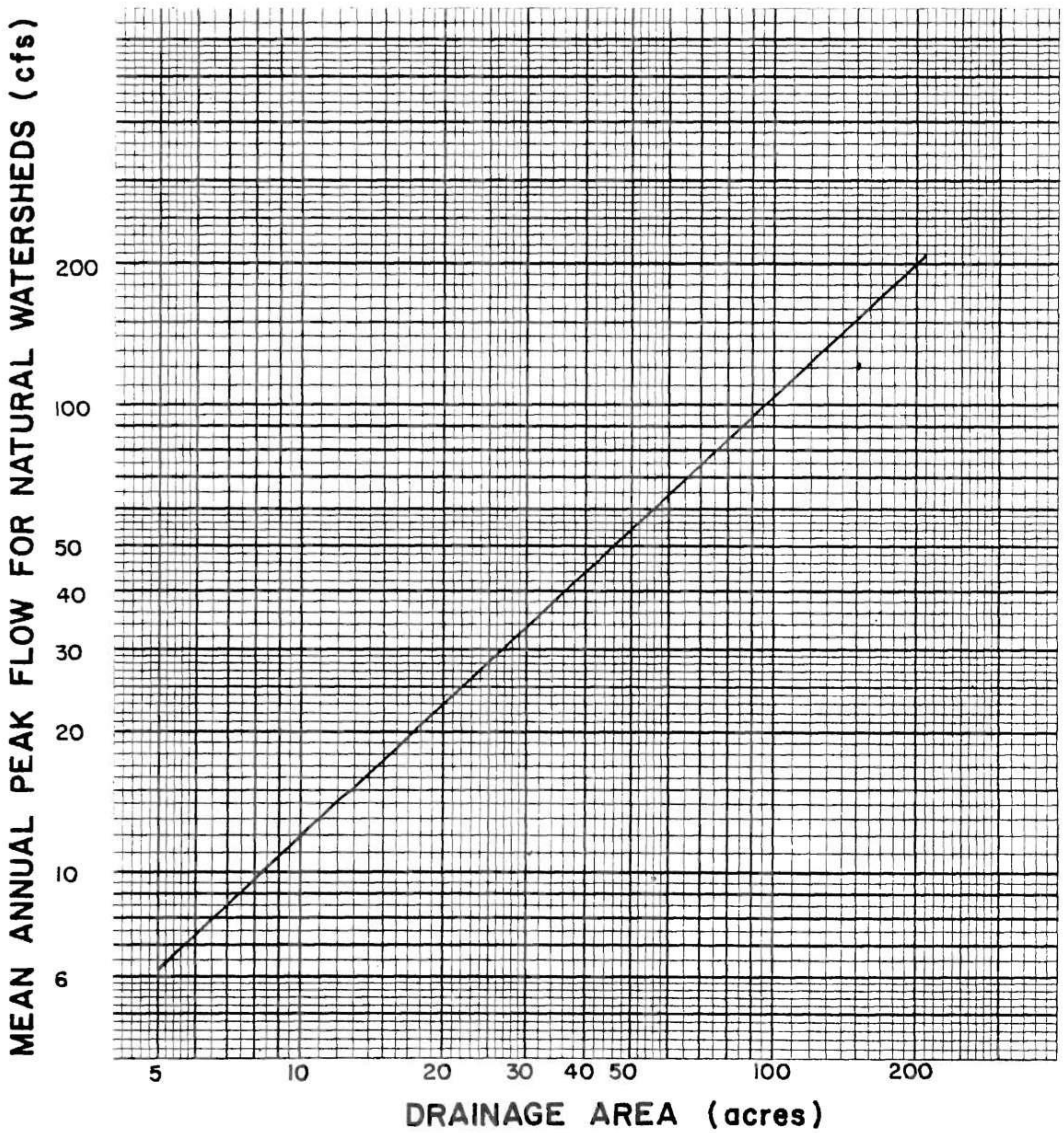

Figure 19. Mean Annual Peak Flow for Small Natural Watersheds vs. Drainage Area.

resulting from this simulation are shown in Table 7 under Comparative Simulation #1.

The second simulation modelled selected watershed sizes as combinations of 13-acre, 10-acre, and 3-acre unit source areas. This simulation was made using the basic stream network of the hypothetical watershed. Referring to Figure 15, source area segments 1, 2, and 9 were combined into a 13-acre source area segment whose runoff discharged laterally into channel segment 57. Source area segments 3, 4, and 10 were combined and discharged laterally into channel segment 58. Source area segments 5, 6, and 11 were combined and discharged laterally into channel segment 59. Source area segments 7 and 8 were combined into a 10-acre source area segment which discharged to dummy segment 60. The same pattern of alteration was repeated for all four symmetric sub-basins of the 208-acre hypothetical watershed. The configuration of source area and channel segments along with 2000-foot reach of the basic stream network was retained. The resulting mean annual peak flows are shown in Table 7 under Comparative Simulation #2.

The third simulation modelled 208 acres as four 49-acre unit source areas plus a 9-acre and a 3-acre source area. Again the basic stream network of the hypothetical watershed was used. Source area segments 1 through 11 were combined into a 49-acre source area segment which discharged laterally into a 1500-foot channel with the hydraulic properties of channel segment 59. This same pattern of alteration was repeated for all four symmetric sub-basins. A 3-acre source area segment discharged laterally to channel segment 95 and a 9-acre source area segment discharged laterally to a 1500-foot channel segment with the hydraulic properties of channel segment 96. The resulting mean annual peak flows are shown in

Table 7. Comparison of Mean Annual Peak Flow Estimates for Natural Watersheds

Drainage Area (AC)	Mean Annual Peak Flow (CFS)				
	Equation 11	Comparative Simulation #1	#2	#3	USGS*
5	6.1	7.7	-	-	11
13	15	17	16	-	19
26	29	30	31	-	29
39	43	40	46	-	36
49	53	48	57	42	42
101	105	82	116	86	64
208	209	139	225	171	99

*Source: Flood-Frequency Analysis for Small Natural Streams in Georgia, USGS, 1976.

Table 7 under Comparative Simulation #3.

Inspection of the data in Table 7, which are plotted in Figure 20, leads to the conclusion that peak flows from UROS simulations are dependent on the manner in which the watershed is segmented. It was originally believed that the more source area segments used to describe a particular watershed, the higher the peak flows would be. The results of these three simulations do not bear this out. Despite the apparent segmentation dependency of UROS simulations, Equation 11 does appear to provide an adequate estimate of expected mean annual peak flow from an undeveloped watershed.

Having selected Equation 11 as the best estimate of expected mean annual peak flows from undeveloped watersheds, a comparison was made with a similar equation recently published by H. G. Golden and McGlone Price (USGS, 1976). Their research produced estimates of expected peak flows for small natural watersheds throughout Georgia. The set of equations given for their Region 2, which includes DeKalb County, are based on the Log-Pearson Type III distribution. Accordingly, the equation for 2-year flows is used for comparison. These flows have an exceedence probability of 50 percent, whereas the mean annual flows from the Gumbel distribution used in UROS have an exceedence probability of 43 percent. This small discrepancy is neglected in this comparison. The USGS equation for natural watersheds in Region 2 is:

$$Q_2 = 195\ A^{.60} \tag{12}$$

where Q_2 is the expected peak flow for a 2-year return period and A is the drainage area in square miles.

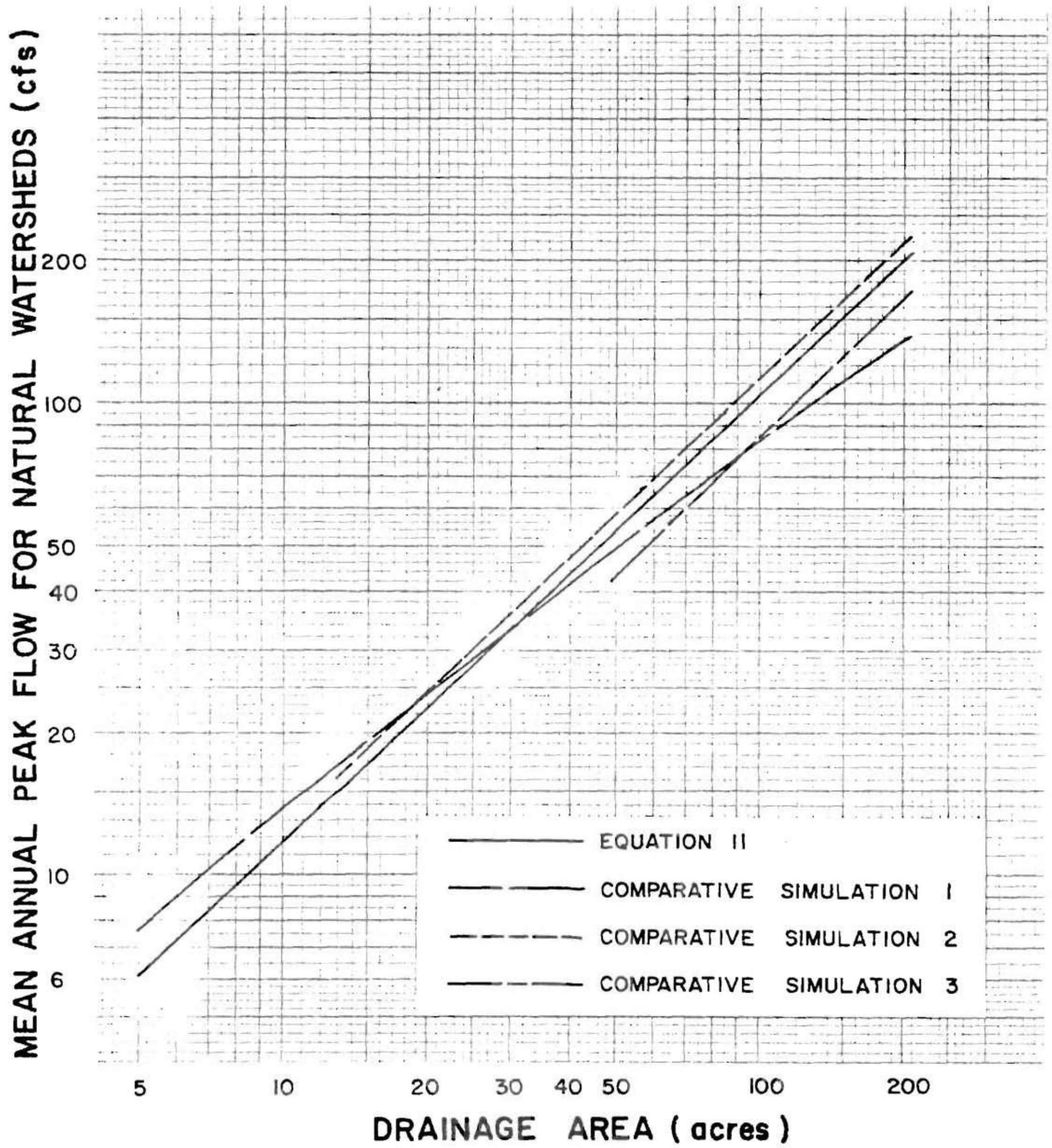

Figure 20. Comparison of Mean Annual Peak Flows from Different Watershed Segmentations.

The USGS equation produced the peak flow estimates given in Table 7 and plotted in Figure 21. Also plotted in Figure 21 are the estimates from Equation 11. The reasons for the large discrepancies may be due, in part, to the size of watersheds on which the equations are based. Golden and Price (1976) pointed out that their method needed to be confirmed for watersheds outside the range of values that they had used. In their paper Conclusion and Recommendation #7 states:

> Users are cautioned that the equations developed herein are applicable to small natural drainage areas of 0.1 to 20 square miles in Georgia. Extrapolation of the equations for use with smaller basins should be checked by other methods and judgment exercised in the application.

Since the USGS equation is based primarily on flows from much larger basins than considered in this investigation, the difference in estimates is not surprising. Additionally, Conclusion and Recommendation #8 states:

> Small-stream gaging stations were not located in the western Piedmont, southwest Georgia, or in the near-coastal areas because of operational and funding limitation. Flood data should be acquired in these areas to verify the use of the flood estimating equations shown herein, or to provide the necessary data for development of applicable equations.

Only 2 out of the 29 gaged watersheds in Region 2 are within 35 miles of the center of DeKalb County. Most are in the extreme eastern part of the Piedmont Area. This fact may also help to explain the difference between the two equations.

Estimates of expected peak flows based on simulated data from UROS have been shown to be somewhat dependent on the manner in which the watershed was segmented for simulation; however, in the absence of gaged stream flow data for small watersheds, the simulated data appear to offer the best available estimate on expected peak flows. Accordingly, Equation 11 is used in the present study as the estimator of expected mean annual

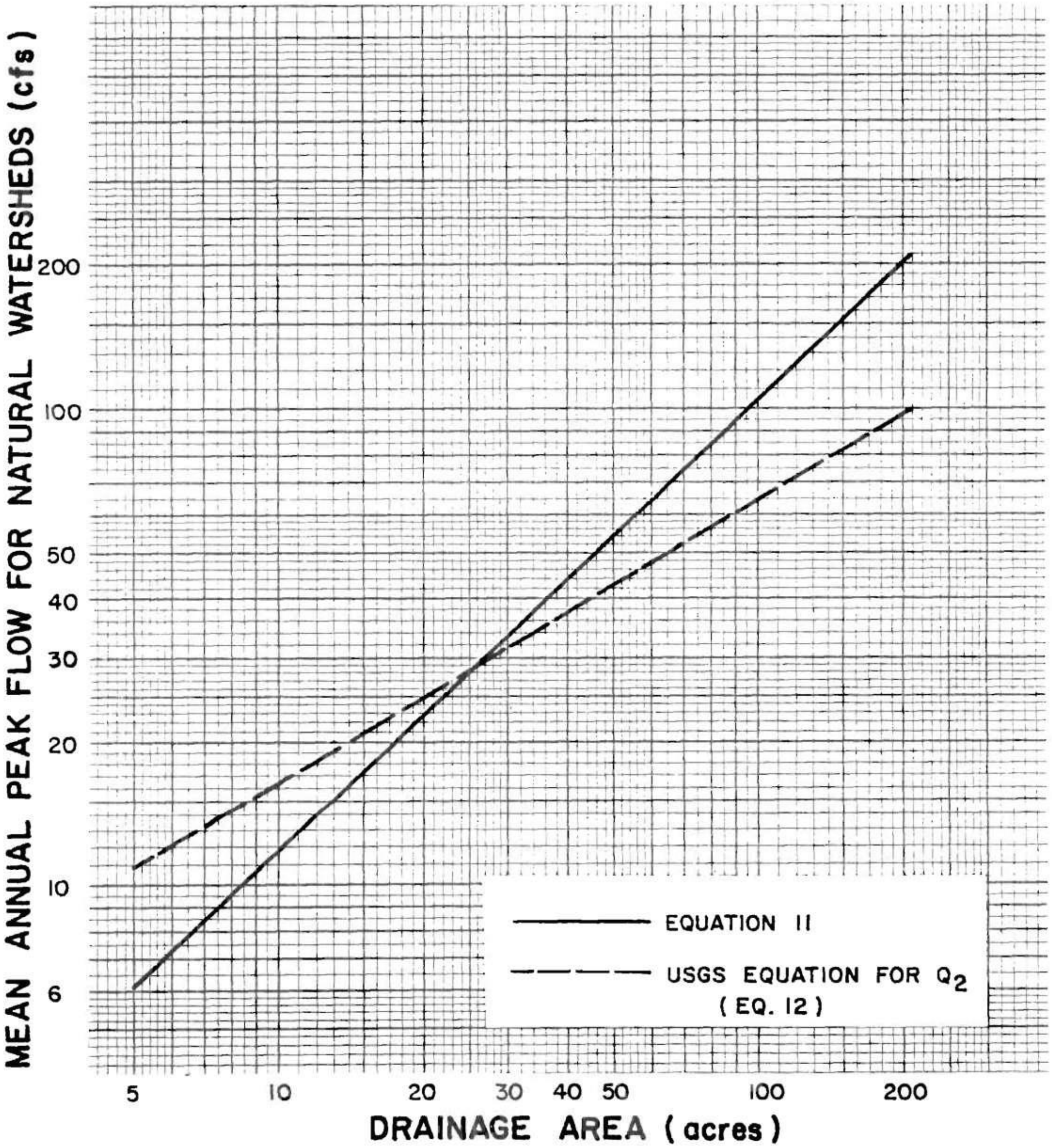

Figure 21. Comparison of Mean Annual Peak Flow Estimate with USGS Equation.

peak flows for small natural watersheds in DeKalb County.

Ratio of Peak Flood Flows to Mean Annual Peak Flow for Natural Watersheds

Based on the five regression equations for specific return periods, Equations 6-10, the ratio of the expected peak flow for any return period, Q_T, to the mean annual peak flow, $\overline{Q}$ from Equation 4, was determined for small natural watersheds. Such an array of ratios allows for the establishment of a frequency-peak flow relationship for undeveloped watersheds once the mean annual peak flow has been estimated. The ratios are given in Table 8 and their relationship to return period is plotted in Fi ure 22. Also shown in Table 8 are ratios from two other sources. One set of ratios was developed from Golden and Price (1976) and is with respect to the two-year return period event. The other set of ratios, given by the U.S. Army Corps of Engineers (1970), is for use with the mean annual peak flow.

The variation among the three sets of ratios may be attributed to the fact that the current investigation deals with watersheds less than 1/3 square mile in DeKalb County. The other methods deal with a larger range of watershed sizes occurring throughout various hydrologic provinces of Georgia. The USGS and Corps of Engineer ratios may also be incorporating a safety factor, due to the uncertainty of estimating expected peak flows. This would tend to create relatively higher ratios.

Method for Determining Expected Peak Flows from Small Watersheds

The purpose of this research was to develop a simple method of determining the effects of urbanization on expected peak flow rates for small watersheds in DeKalb County, Georgia. This was accomplished through

Table 8. Comparison of Ratios of Expected Peak Flow to Mean Annual Peak Flow for Natural Watersheds

Return Period	Equations from Table 6 $Q_T/\overline{Q}$	USGS Q_T/Q_2	Corps of Engineers $Q_T/\overline{Q}$
5	1.4	1.7	1.4
10	1.8	2.3	1.8
25	2.3	3.1	2.5
50	2.6	3.7	3.0
100	3.0	4.4	3.6

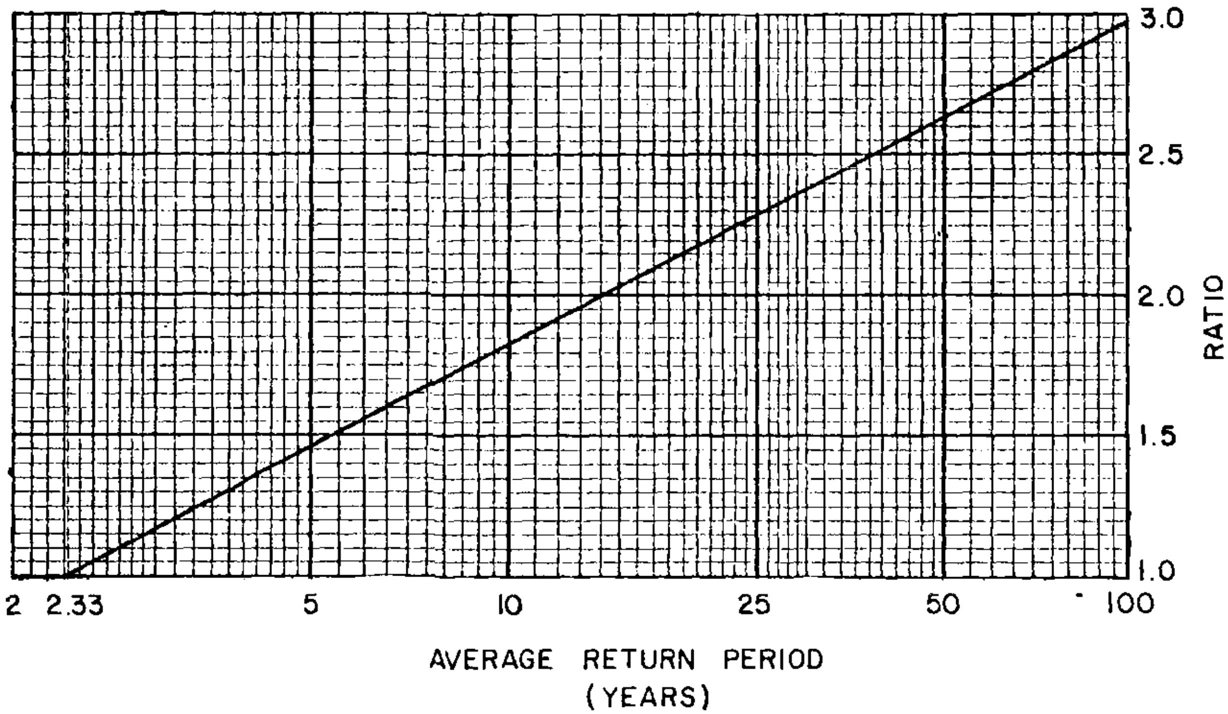

Figure 22. Ratio of Expected Peak Flow to Mean Annual Peak Flow for Small Natural Watersheds vs. Return Period.

preparation of a semi-graphical solution of the regression equations developed in this investigation (Equations 4-10).

As previously demonstrated, Figure 19 or Equation 11 provides an estimate of the expected mean annual peak flow for undeveloped watersheds between 5 and 208 acres. The ratio from Figure 22 when multiplied by the mean annual peak flow yields an estimate of the frequency-peak flow relationship for an undeveloped watershed. Inspection of the regression equations in Table 6 reveals that the exponents of the terms for percent imperviousness, road density, and percent of channels hydraulically modified are different for different return periods; accordingly, a different graphical solution utilizing these three urbanization terms was prepared for each return period. The simple method for determining expected peak flows from small watersheds involves five steps, which are illustrated by an example given in Appendix B. As presented, this simple method may be used in two different applications. In one instance it may be used as an estimator of design flows for small watersheds much in the same manner that the Rational Method or other empirical equations have been used in the past. The benefits of the method as presented are: (1) it avoids the subjective selection of runoff coefficients; (2) it avoids the determination of a time of concentration; (3) it is based on long term synthesized flood data rather than rainfall data; (4) it incorporates the separate effects of imperviousness and sewering; and (5) it provides results that are consistent from one application to another.

It may also be used as an estimator of the hydrologic response of a watershed as simulated by UROS. This application affords the engineer the opportunity to develop inexpensive preliminary flow information that will not differ greatly from the flow information generated from modelling with UROS at a later stage during design and analysis. A comparison of the simple method with UROS simulations of actual DeKalb County watersheds was undertaken as part of this investigation and is discussed in the next section.

An alternate semi-graphical method to the one presented in Appendix B is presented in Appendix C. It involves determining the effects of urbanization on the mean and standard deviation of the annual peak flow series.

Accuracy of Method

To investigate the capability of the regression equations (Equations 4-10) to accurately estimate a frequency-peak discharge relationship generated with UROS, comparisons were made with simulations of several small watersheds. Thirty watershed configurations were selected from the Echo Branch Sub-basin described previously, and flood frequencies were determined by full simulation and by the simple method. None of these configurations contained storage segments and all channels were in natural conditions.

When comparing the simple method with UROS, certain considerations must be made. For example, if the watershed segmentation does not include modelling road gutters as channel segments, then the road density should be disregarded in applying the simple method. The user should enter the Urbanization Factor Charts (Appendix B) at Road Density equal to zero. However, if the simplified method is being used to develop flow information for some other purpose, then all factors should be included since, as this investigation has shown, road density has some affect on expected peak flows from small watersheds.

Table 9 is a comparison of estimates from the regression equation (Equation 4) and simulation results using UROS for mean annual peak flow values for Echo Branch Sub-basin. Figure 23 is a plot of estimated mean annual peak flow against simulated mean annual peak flow. The average error in estimating is -0.4 percent and the standard deviation of the error is 9.3 percent. Inspection of these data reveals that two-thirds of the estimates are within 10 percent of the simulated values. Inspection also reveals that the simple method tends to underestimate lower mean annual flows (simulated) and overestimate higher mean annual flows, with the transition point appearing to be between 50 to 70 cfs. This apparent correlation may be attributed to the fact that lower mean annual flow values have been simulated from single source area segments. As was shown in Figure 20, this underestimation can be expected. Similarly, Figure 20 also indicates that for larger watersheds the simulation results are segmentation dependent.

Comparisons of estimated and simulated values were also made for the Echo Branch sub-basin watersheds for the standard deviation of the annual peak flow series and for the expected 100-year return period flows. Results of these comparisons are shown in Tables 10 and 11, and Figures 24 and 25. The average error in estimating the standard deviation of the annual peak flow series is +4.2 percent. The average error in estimating the expected 100-year peak flow is +1.5 percent. These two sets of errors in estimating have standard deviations of 9.1 percent and 8.5 percent respectively. From these comparisons, it is concluded that flood frequencies based on the regression equations from this investigation are approximately equal to the frequency-peak discharge relationship resulting

Table 9. Comparison of Estimated and Simulated Mean Annual Peak Flows

Drainage Area (AC)	Impervious Fraction	Simulated $\overline{Q}$	Estimated $\overline{Q}$	$\frac{\text{Est.-Sim.}}{\text{Sim.}}$
16.2	.28	30.0	25.7	-14%
28.8	.07	35.9	34.6	- 4%
42.2	.15	54.9	54.8	0%
47.1	.11	56.5	57.9	+ 2%
28.8	.11	38.5	36.3	- 6%
33.1	.17	46.7	44.6	- 4%
45.0	.146	57.6	57.9	+ 1%
89.3	.129	99.0	108.8	+10%
73.8	.132	90.6	91.1	+ 1%
163.1	.130	189.3	192.9	+ 2%
196.2	.137	194.9	231.8	+19%
33.4	.12	43.9	42.3	- 4%
52.6	.16	66.3	68.3	+ 3%
32.8	.18	47.3	44.7	- 5%
26.0	.12	35.9	33.4	- 7%
31.0	.16	43.6	41.4	- 5%
25.0	.18	37.9	34.5	- 9%
59.4	.12	72.7	73.1	+ 1%
83.6	.16	95.0	106.1	+12%

Table 9. (Continued

Drainage Area (AC)	Impervious Fraction	Simulated $\overline{Q}$	Estimated $\overline{Q}$	$\frac{\text{Est.-Sim.}}{\text{Sim.}}$
57.8	.18	71.7	76.5	+ 7%
200.5	.154	238.2	241.9	+ 2%
12.1	.10	18.4	15.8	-14%
14.9	.95	59.0	45.5	-23%
54.4	.19	72.3	73.1	+ 1%
85.7	.15	95.4	107.3	+12%
81.4	.316	111.4	123.9	+11%
167.1	.213	192.5	222.5	+16%
19.7	.13	29.0	26.0	-10%
34.1	.13	45.2	43.7	- 3%
53.8	.13	69.6	67.3	- 3%

Avg. = -0.4%

Stand. Dev. = 9.3%

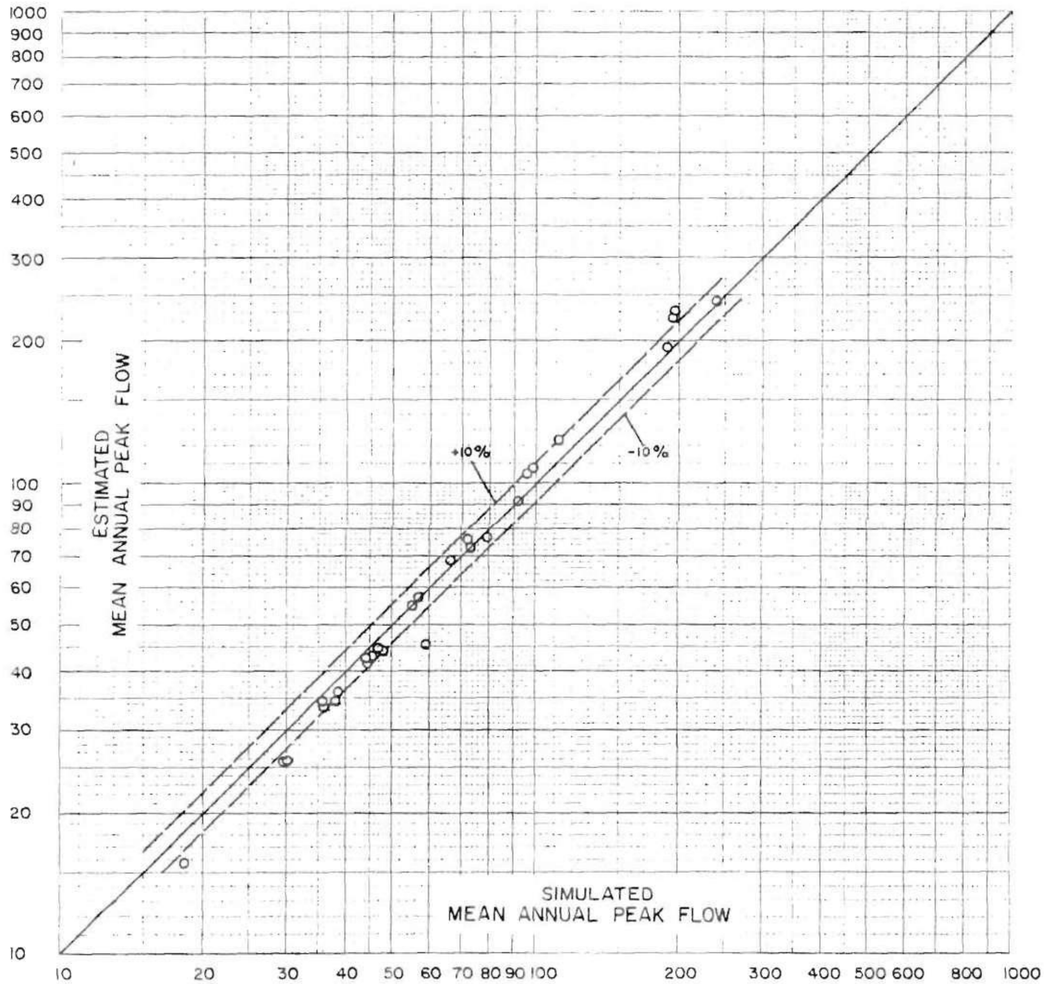

Figure 23. Comparison of Estimated and Simulated
Mean Annual Peak Flows.

Table 10. Comparison of Estimated and Simulated Standard Deviations of the Annual Peak Flow Series

Drainage Area (AC)	Impervious Fraction	Simulated Standard Deviation	Estimated Standard Deviation	Est.-Sim. / Sim.
16.2	.28	13.8	12.8	- 7%
28.8	.07	19.9	20.3	+ 2%
42.2	.15	28.6	30.0	+ 5%
47.1	.11	30.5	32.9	+ 8%
28.8	.11	20.5	20.5	0%
33.1	.17	23.8	24.0	+ 1%
45.0	.146	30.2	31.8	+ 5%
89.3	.129	52.8	60.9	+15%
73.8	.132	49.3	50.3	+ 3%
163.1	.130	101.7	108.4	+ 7%
196.2	.137	99.0	129.7	+31%
33.4	.12	23.3	23.7	+ 2%
52.6	.16	34.4	37.2	+ 8%
32.8	.18	23.9	23.9	0%
26.0	.12	19.0	18.7	- 2%
31.0	.16	22.4	22.4	0%
25.0	.18	19.0	18.4	- 3%
59.4	.12	39.0	41.1	+ 5%
83.6	.16	53.2	57.9	+ 9%

Table 10. (Continued)

Drainage Area (AC)	Impervious Fraction	Simulated Standard Deviation	Estimated Standard Deviation	Est.-Sim. / Sim.
57.8	.18	39.7	41.0	+ 3%
200.8	.154	131.7	133.4	+ 1%
12.1	.10	24.0	20.2	-16%
14.9	.95	9.7	8.9	- 8%
54.4	.19	36.4	38.9	+ 7%
85.7	.15	50.2	59.0	+18%
81.4	.316	53.1	61.2	+15%
167.1	.213	97.8	115.9	+19%
19.7	.13	23.8	24.3	+ 2%
34.1	.13	15.1	14.4	- 5%
53.8	.13	37.2	37.6	+ 1%
				Avg. = +4.2%
				Stand. Dev. = 9.1%

Table 11. Comparison of Estimated and Simulated 100-year Return Period Peak Flows

Drainage Area (AC)	Impervious Fraction	Simulated Q_{100}	Estimated Q_{100}	$\frac{\text{Est.-Sim.}}{\text{Sim.}}$
16.2	.28	73	66	-10%
28.8	.07	98	98	0%
42.2	.15	145	148	+ 2%
47.1	.11	152	160	+ 5%
28.8	.11	103	100	- 3%
33.1	.17	122	119	- 2%
45.0	.146	152	157	+ 3%
89.3	.129	265	298	+12%
73.8	.132	245	249	+ 2%
163.1	.130	508	530	+ 4%
196.2	.137	506	635	+25%
33.4	.12	117	116	- 1%
52.6	.16	174	184	+ 6%
32.8	.18	122	119	- 2%
26.0	.12	95	91	- 4%
31.0	.16	114	111	- 3%
25.0	.18	98	92	- 6%
59.4	.12	195	201	+ 3%
83.6	.16	262	286	+ 9%

Table 11. (Continued)

Drainage Area (AC)	Impervious Fraction	Simulated Q_{100}	Estimated Q_{100}	$\frac{\text{Est.-Sim.}}{\text{Sim.}}$
57.8	.18	196	204	+ 4%
200.8	.154	651	657	+ 1%
12.1	.10	134	110	-18%
14.9	.95	49	44	-10%
54.4	.19	186	194	+ 4%
85.7	.15	253	291	+15%
81.4	.316	275	282	+ 1%
167.1	.213	499	585	+17%
19.7	.13	120	119	- 1%
34.1	.13	76	71	- 7%
53.8	.13	186	184	- 1%
				Avg. = +1.5%
				Stand. Dev.= 8.5%

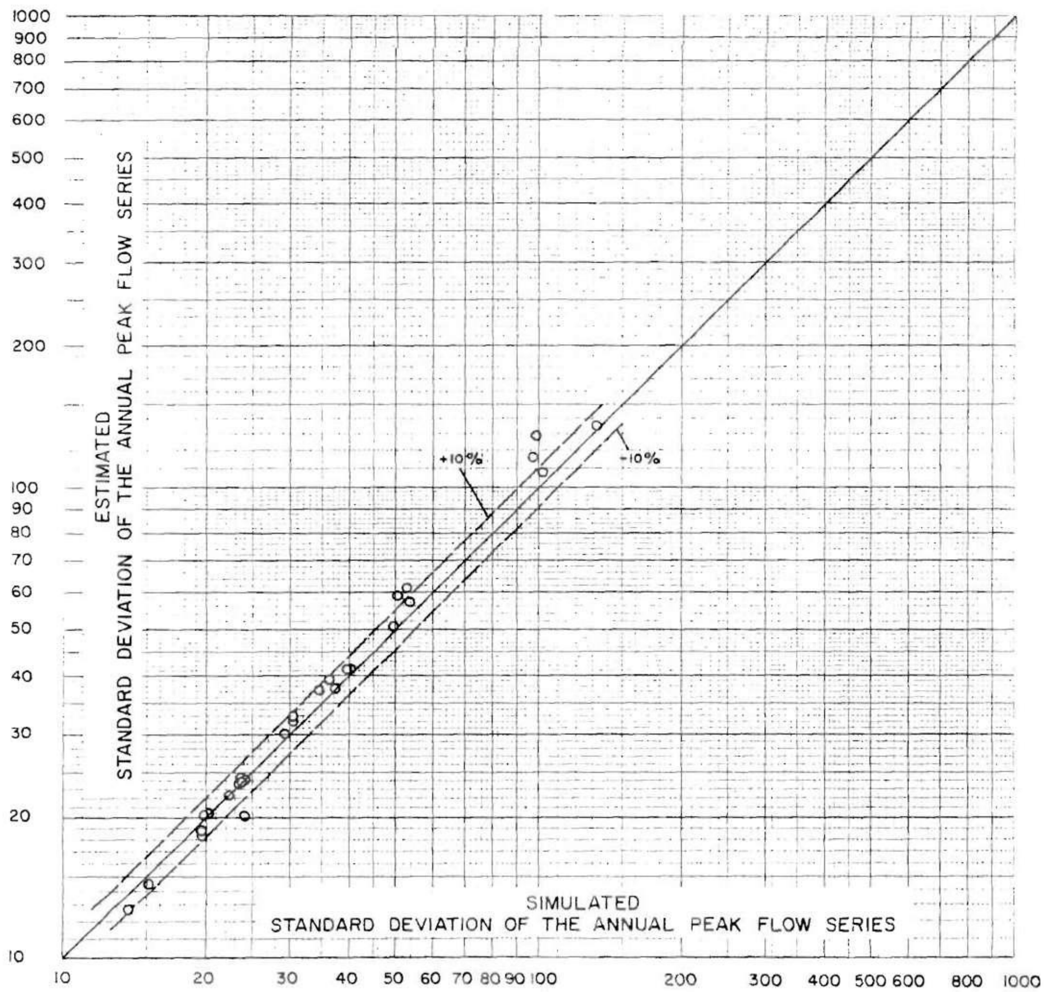

Figure 24. Comparison of Estimated and Simulated Standard Deviations of the Annual Peak Flow Series.

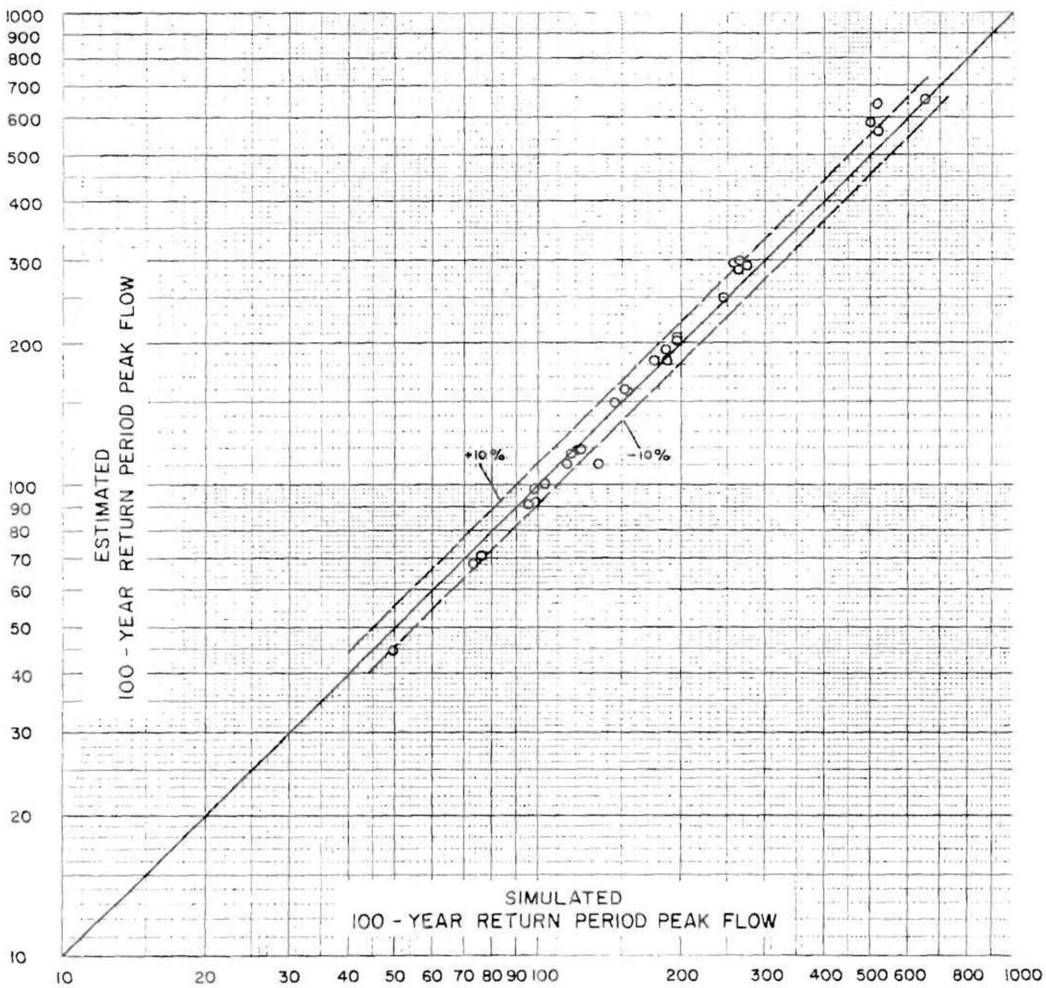

Figure 25. Comparison of Estimated and Simulated 100-year Return Period Peak Flows.

from a UROS simulation of a small watershed. It should be emphasized that the simulation results on which this comparison is based are from studies of actual watersheds rather than from the hypothetical watershed used for the development of the regression equations.

Comparison with Results of James and Lumb

James and Lumb (1975) made estimates of the separate effects of imperviousness and channelization on expected peak flow values. Their estimates are based on simulations of the Warren Creek sub-basin with minor modifications to the watershed configuration so as to exclude all storage segments. This 1060-acre watershed, which has an average imperviousness of 39 percent, is a tributary to the North Fork of Peachtree Creek.

To investigate the relationship of flood peaks to impervious area, simulations of Warren Creek sub-basin with natural channels were made by James and Lumb for three watershed land-use conditions (zero imperviousness, existing imperviousness and projected 85 percent imperviousness). For 12 flow points in the sub-basin, the ratios of mean annual peak flow under developed conditions to mean annual peak flow under natural conditions were determined. A regression analysis of ratios from these 12 data points yielded the equation

$$M = 1.0 + 2.3I \qquad (14)$$

where M is the ratio, and I is the decimal fraction of imperviousness. Based on the current simulation study the expected mean annual peak flow ratio is

$$M = \left[(1 + I)^{1.657}\right]^{\left[\log\ (1+I)\right]^{0.1}} \qquad (15)$$

where M is the ratio and I is the decimal fraction of impervious area.

Figure 26 and Table 12 show a comparison between the ratios from this investigation and the ratios determined by James and Lumb. The probable reason for the difference is suggested by the statement from James and Lumb, "One would expect the effect of impervious area to be smaller in larger basins since channel storage increases geometrically with basin size . . .". The Warren Creek watershed and its sub-basins are generally larger than the hypothetical watershed and its sub-basins; however, the latter was simulated with more channel segments and thus more channel storage. The Warren Creek watershed was simulated with an average stream density of 1.8 miles per square mile. The hypothetical watershed was simulated with an average natural stream density of over 4 miles per square mile (Table 1). Based on the statement by James and Lumb, the ratios from Warren Creek sub-basin might be expected to be lower than those from the hypothetical watershed because of watershed sizes. Since this is not reflected in the comparison, it is concluded that the effects of channel storage do influence the relationship of flood peaks to impervious areas to some degree. Nevertheless, the relationship of peak flood ratios to watershed imperviousness as presented by this investigation tends to substantiate that proposed by James and Lumb and is offered for general use in estimating flood peak ratios for small watersheds.

The relationship of peak flood flows to channelization was investigated by James and Lumb by simulating improved channels as concrete lined rectangular channels that completely contained all flows. For the full Warren Creek watershed, all-channels-improved and the downstream-half-of-the-channels-improved were the two conditions simulated. The resulting relationship for expected mean annual peak flows to channelization is

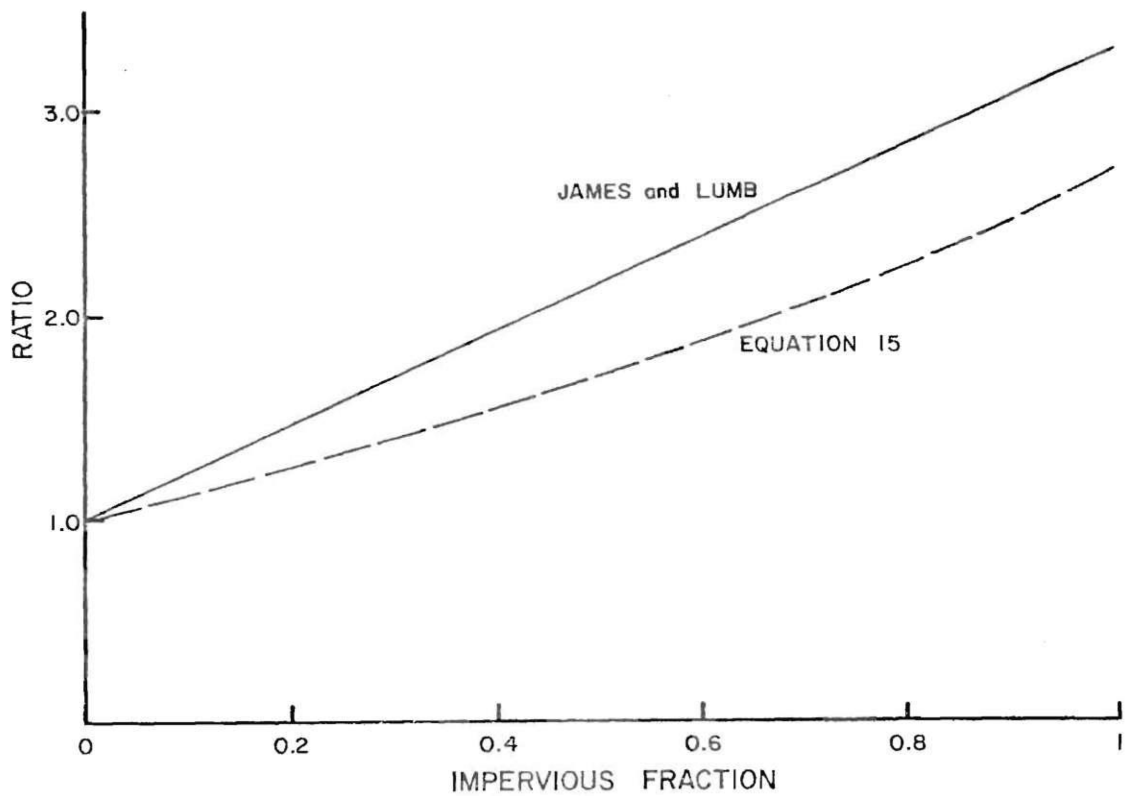

Figure 26. Comparison of Ratios of Expected Mean Annual Peak Flows for Urban Watersheds to Expected Mean Annual Peak Flows for Natural Watersheds as a Function of Imperviousness (after James and Lumb).

Table 12. Comparison of Ratios of Expected Mean Annual Peak Flows for Urban Watersheds to Expected Mean Annual Peak Flows for Natural Watersheds as a Function of Imperviousness

IMP	(Equation 14) $M = 1.0+2.3I$	(Equation 15) $M = \left[(1+I)^{1.657}\right]^{\left[\log(1+I)\right]^{0.1}}$
0	1.00	1.00
.2	1.46	1.26
.4	1.92	1.58
.6	2.38	1.94
.8	2.84	2.34
1.0	3.30	2.77

M = ratio

I = decimal fraction of impervious area

shown in Figure 27.

The effects of channelization were investigated using the hypothetical watershed by simulating improved channels as slightly enlarged channels with rip-rap bank stabilization and no reduction of overbank floodplain storage. The influence of channelization on expected peak floods is reflected in the regression equations by the term for the percent of hydraulically modified channels. As a result, for the expected mean annual peak flow, the ratio is given by the equation:

$$M = (1 + HM)^{0.116} \tag{16}$$

where M is the ratio to natural channels, and HM is the decimal fraction of channel length that is hydraulically modified. This relationship of flood peak to channelization is also shown in Figure 27. The difference between the relationship proposed by this investigation and that proposed by James and Lumb may be attributed to the difference in the manner in which improved channels were simulated, and also to the difference in the size of the watersheds investigated.

To investigate the influence of watershed size on channelization, two investigations were made. First, of the 12 sub-basins in the Warren Creek watershed, six are in the range of drainage areas investigated using the hypothetical watershed. From these six sub-basins, ratios of expected mean annual peak flow for fully channelized watersheds to expected mean annual peak flows for natural channel watersheds were determined. This was done for each of three conditions of impervious area (zero imperviousness, existing imperviousness, and projected 85 percent imperviousness).

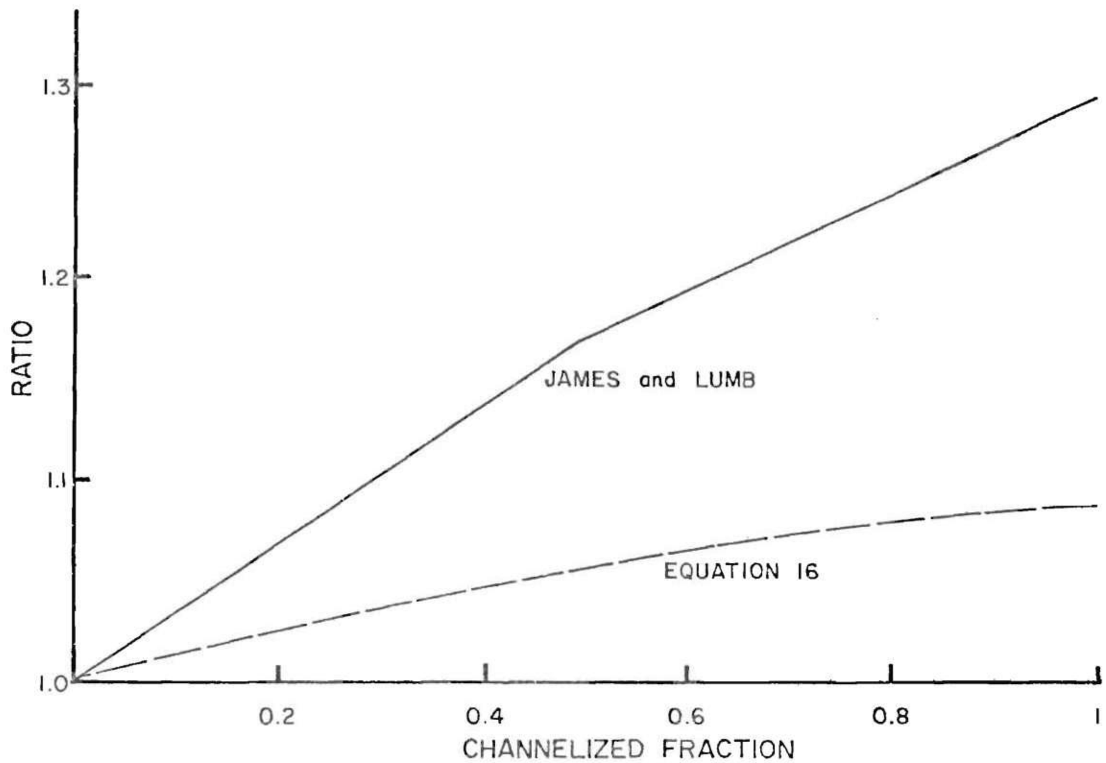

Figure 27. Comparison of Ratios of Expected Mean Annual Peak Flows for Watersheds with Hydraulically Modified Channels to Expected Mean Annual Peak Flows for Natural-channel Watersheds as a Function of Channel Length Modified (after James and Lumb).

The average of these 18 ratios, shown in Table 13, is 1.14, which compares favorably with the ratio 1.08 from Equation 16 based on the hypothetical watershed. The ratio for the larger 1,060-acre Warren Creek sub-basin is 1.30. The second investigation involves the conclusion by James and Lumb that the effects of channelization do in fact increase with drainage area. To verify this, ratios of expected mean annual flow for fully channelized sub-basins to expected mean annual peak flow for natural-channel sub-basins were determined for sub-basin sizes of 13, 26, 39, 49, 101, and 208 acres (Table 14). A regression analysis was performed and yielded the relationship shown in Figure 28. Also shown is the relationship proposed by James and Lumb. Regardless of any variation in the suggested magnitude of the effects of channelization, this investigation and that by James and Lumb do conclude that for watersheds in the range investigated, (1) the effects of channelization are considerably less than the effects of impervious area, and (2) the effects of channelization do increase with drainage area.

Effects of Urbanization

Leopold proposed that the effects of urbanization on expected peak flow rates are a function of the percent of impervious area and the percent of area served by storm sewers (Leopold, 1968). Since storm sewer systems are not extensive in DeKalb County it was assumed in this investigation that the "sewering effect" was a function of the length of road gutters and the length of hydraulically modified channels in the watershed.

Leopold's chart for the effects of urbanization displays the ratios of expected peak flows for developed conditions to the expected peak flows for undeveloped conditions. His chart is for mean annual flows and drainage

Table 13. Ratios of Expected Mean Annual Peak Flows for Fully Channelized Watersheds to Expected Mean Annual Peak Flows for Natural Channel Watersheds for the Warren Creek Watershed

% Impervious Area	Drainage Area (Acres)					
	34	46	57	151	217	218
0	1.09	1.17	1.07	1.04	1.09	1.06
Existing	1.14	1.20	1.07	1.03	1.09	1.06
85	1.24	1.26	1.35	1.19	1.21	1.18

Table 14. Ratios of Expected Mean Annual Peak Flows for Fully Channelized Watersheds to Expected Mean Annual Peak Flows for Natural Channel Watersheds for the Hypothetical Watershed

% Impervious Area	Drainage Area 13	26	39	49	101	208
0	1.03	1.04	1.06	1.04	1.06	1.09
.20	1.05	1.06	1.08	1.06	1.07	1.11
.40	1.05	1.07	1.09	1.06	1.08	1.13
1.00	1.06	1.09	1.11	1.08	1.11	1.17

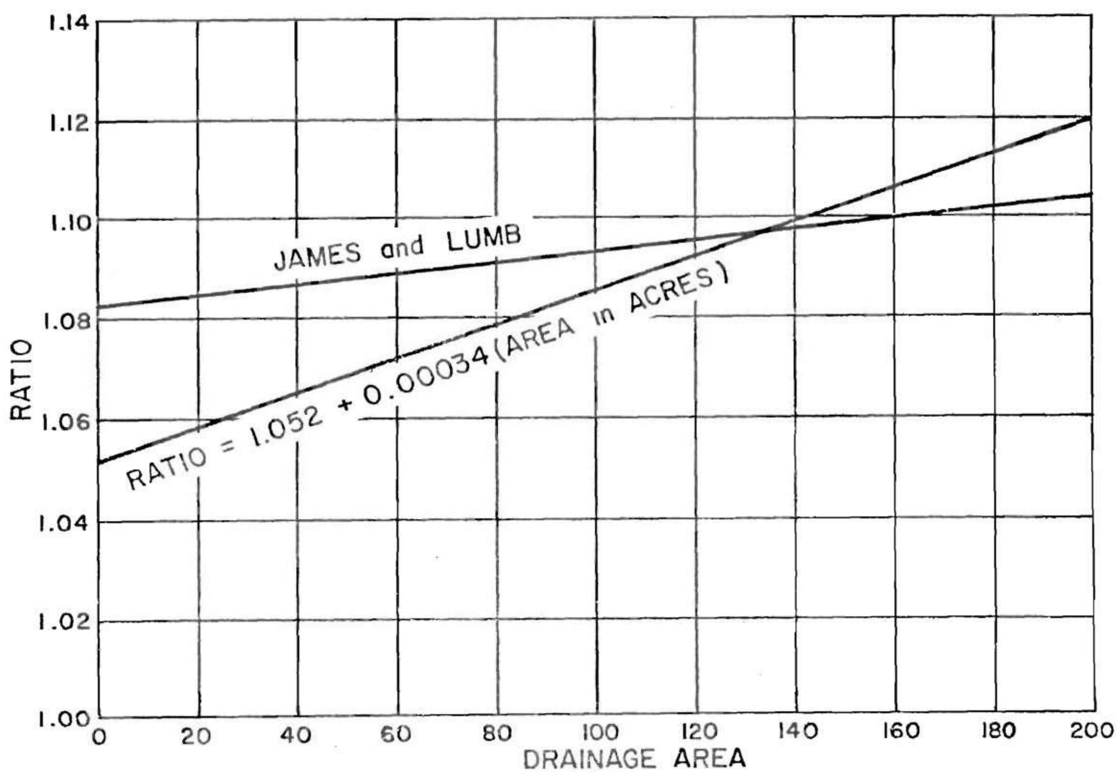

Figure 28. Comparison of Ratios of Expected Mean Annual Peak Flows of Fully Channelized Watersheds to Expected Mean Annual Peak Flows for Natural-channel Watersheds as a Function of Drainage Area.

areas of one square mile. Although Leopold presents only the one chart shown in Figure 29, he states that the ratios should be different for different drainage area sizes and different return periods.

The results of this investigation can be compared with Leopold's chart if certain assumptions are allowed. It is assumed that X "percentage of area served by storm sewerage" on Leopold's chart can be equated to X percent of channels hydraulically modified combined with X percent of the road density of a typical developed watershed. This developed watershed road density was taken as 10 miles per square mile. For example, if 30 percent of the area is "served by storm sewers" according to Leopold's chart, it is assumed that 30 percent of the channels are hydraulically modified and the road density is 0.30 x 10 mi/sq.mi. (or 3 mi/sq.mi.). The resulting chart is shown in Figure 30. Comparison of Figure 30 with Leopold's chart, Figure 29, reveals two important points:

1) the maximum ratio of urbanized peak flows to non-urbanized peak flows differs by a factor of approximately two; and
2) although the general shape of the curves is similar, curves in Figure 30 indicate less influence by "storm sewering" than do those in Figure 29.

From this comparison it is concluded that Leopold's chart for the effects of urbanization on peak flows is not strictly applicable to small watersheds in DeKalb County, Georgia.

Stankowski proposed that the effect of urbanization on peak flows is a function of the percent of impervious cover and the return period (Stankowski, 1974). His conclusions are based on multiple regression equations generated from data on 103 gaging stations in New Jersey. These

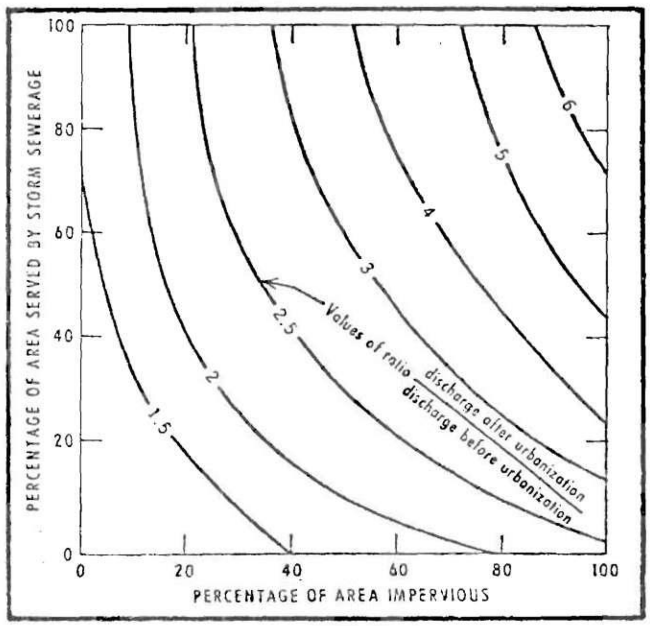

Figure 29. Leopold's Chart for Determining the Effects of Urbanization (Leopold, 1968).

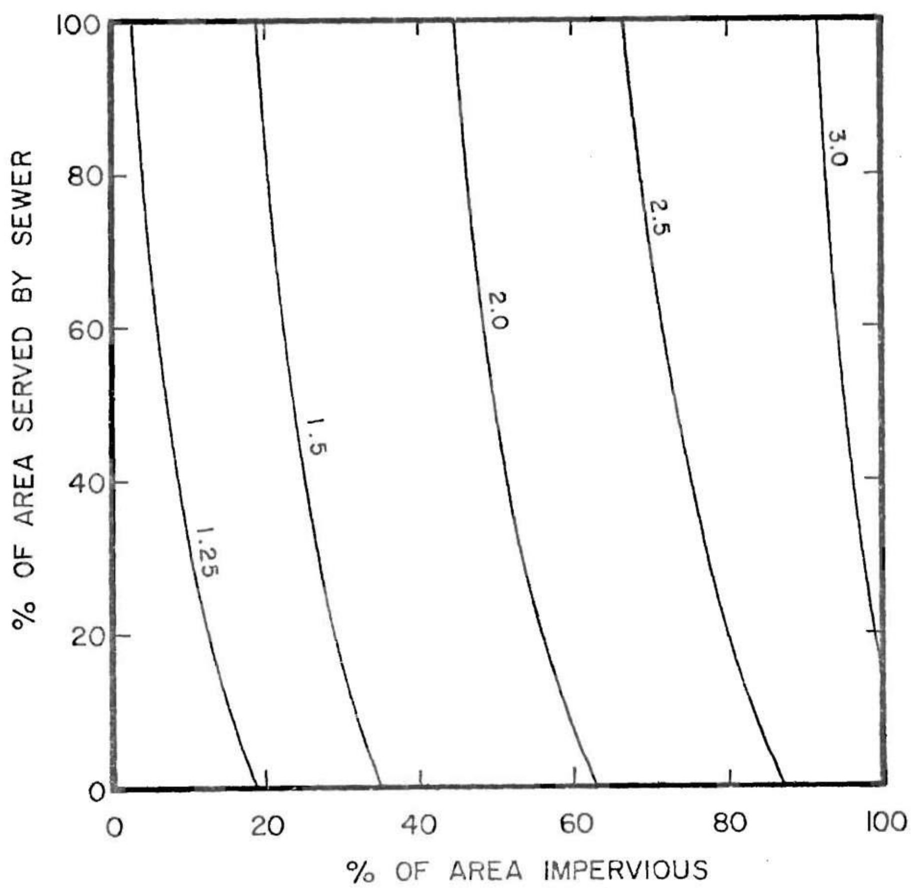

Figure 30. Ratio of Expected Mean Annual Peak Flows for Urbanized Watersheds to Expected Mean Annual Peak Flow for Natural Watersheds Based on Simulation Results with Qualifying Assumptions (after Leopold).

equations are recommended for use on drainage areas from 1 to 1000 square miles. Table 15 shows the state-wide averages of the ratios of flows after urbanization to flows before urbanization as presented by Stankowski.

A comparison of Stankowski's table with the results of the present investigation can be made if changes to Equations 4-10 are made. If the terms for road density and channel modification are dropped from the equations, the entire effect of urbanization is reflected in the term for percent of impervious area. Table 15 contains in parentheses the results of this assumption. Inspection of Table 15 reveals:

1) both sets of ratios diminish with increasing return period, but not at the same rate;
2) both sets of ratios increase with increasing percent of area impervious, but not to the same value; but,
3) there is good agreement between ratios at the 50-year and 100-year return period for the 80 percent impervious area index.

Based on this comparison it is concluded that Stankowski's ratios are not strictly applicable to DeKalb County. These variations in ratios may be attributable to the difference in watershed sizes, climate, and soils in the two states and do not necessarily indicate an inherent fault in either of the proposed relationships.

Comparison with Sauer-Golden Method

By combining the results of previous research, Golden proposed a method for estimating the frequency-peak flow relationships of urbanized watersheds in the metropolitan Atlanta area (Golden, 1977). The method, referred to in this paper as the Sauer-Golden method (or equations), was

Table 15. Increase in Peak Discharge as a Result of Urbanization (After Stankowski)

Recurrence Interval (years)	Ratio of discharge after urbanization to discharge before urbanization				
	Index of manmade impervious cover (percent)				
	1	10	25	50	80
2,2.33*	1.0(1.0)*	1.8(1.15)*	2.2(1.35)*	2.6(1.75)*	3.0(2.35)*
5	1.0(1.0)	1.6(1.10)	2.0(1.25)	2.4(1.65)	2.6(2.15)
10	1.0(1.0)	1.6(1.10)	1.9(1.20)	2.2(1.55)	2.4(2.05)
25	1.0(1.0)	1.5(1.05)	1.8(1.20)	2.0(1.50)	2.2(1.95)
50	1.0(1.0)	1.4(1.05)	1.7(1.20)	1.9(1.50)	2.0(1.90)
100	1.0(1.0)	1.4(1.05)	1.6(1.20)	1.7(1.45)	1.8(1.90)

Source: Magnitude and Frequency of Floods in New Jersey with Effects of Urbanization, Special Report 38, USGS, 1974

*Ratios are for 2.33 year return period. Ratios from this investigation are in parentheses.

compared with the results of this investigation. Selected combinations of watershed size and watershed imperviousness were analyzed using both the Sauer-Golden equations and the UROS equations (Table 6). In applying the Sauer-Golden equation, 50 percent of the area was assumed to be served by storm sewers. This agrees with the assumptions made by Golden (1977). For the UROS equations, a Road Density of 5 miles/sq. mile was assumed along with a modification of 50 percent of the natural channels. These assumptions were necessary to give some basis for a comparison of the two methods which both incorporate the effects of "storm sewering". Comparisons of the two methods could also be made for any degree of sewering, but no others have been undertaken as part of this investigation.

The frequency peak flow relationship generated from both methods for each selected combination of watershed size and imperviousness are shown in Figures 31-35. Based on these results, the percent difference between the Sauer-Golden estimate and the UROS estimate was tabulated. In general, the Sauer-Golden estimates are higher than UROS estimates for watersheds up to approximately 100 acres and lower than UROS estimates for watersheds above approximately 100 acres. Figures 36, 37, and 38 suggest the relationships between estimates produced by the two methods for the 2 year, 10 year and 100 year return period peak flows respectively. Inspection of these figures reveals that good agreement ($\pm$10%) between the two methods exists for only a narrow band of watershed sizes. The boundaries of this band are affected by return period as well as imperviousness. From this comparison it may be concluded that the Sauer-Golden method and the method presented here are not generally compatible for estimating frequency-peak discharge relationships for small watersheds.

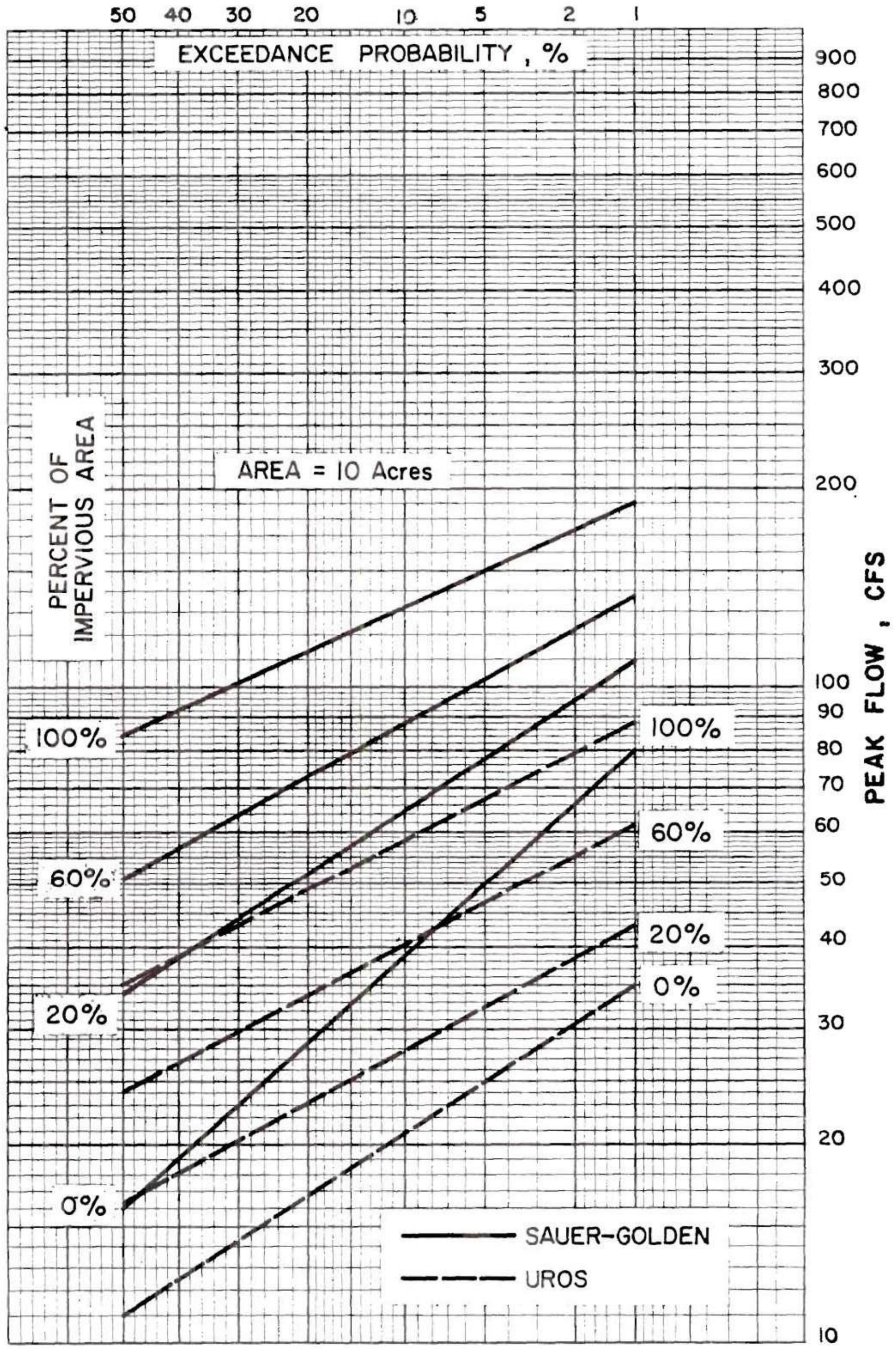

Figure 31. Comparison of Frequency - Peak Discharge Relationships for 10-acre Watersheds.

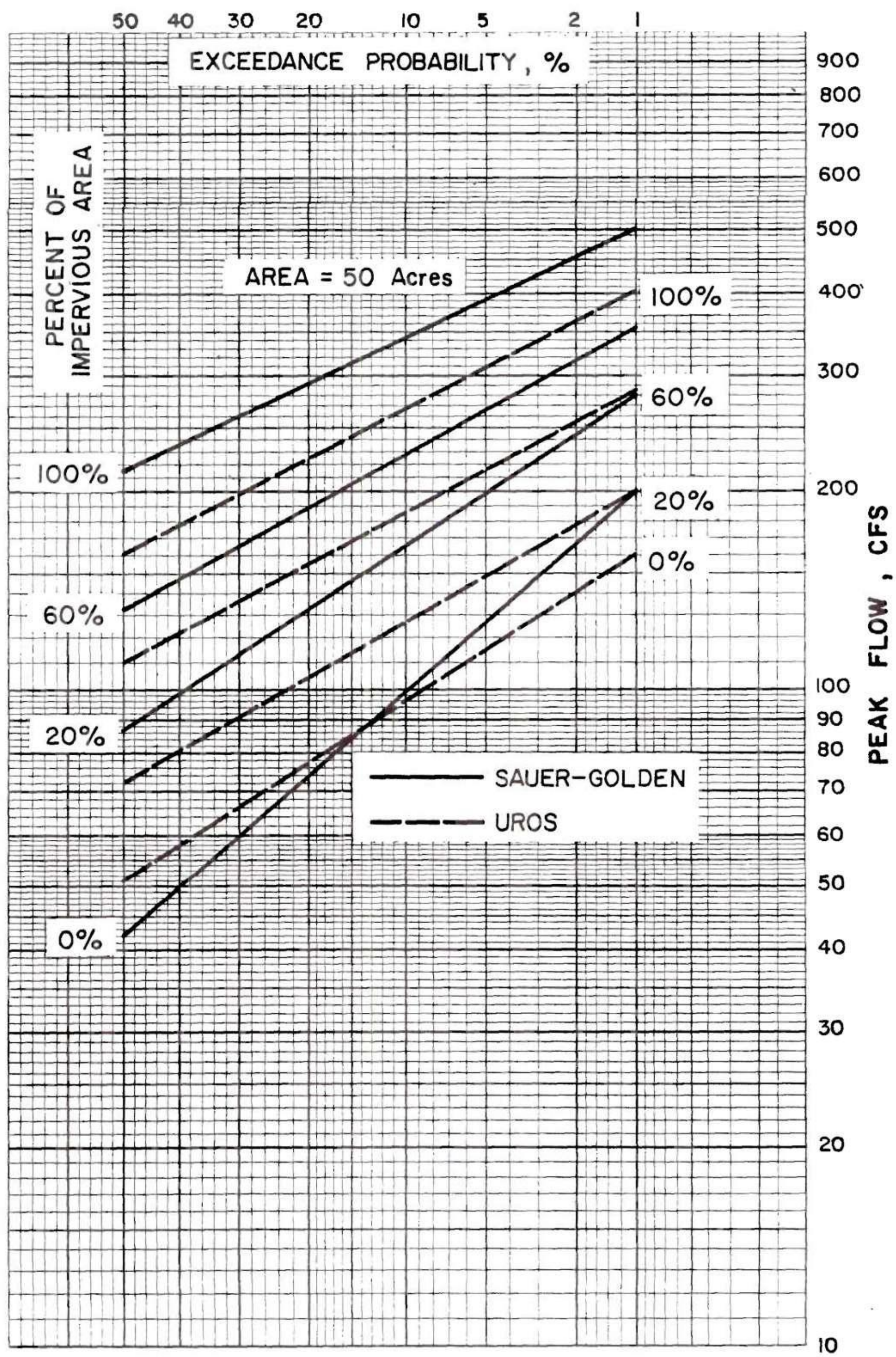

Figure 32. Comparison of Frequency - Peak Discharge Relationships for 50-acre Watersheds.

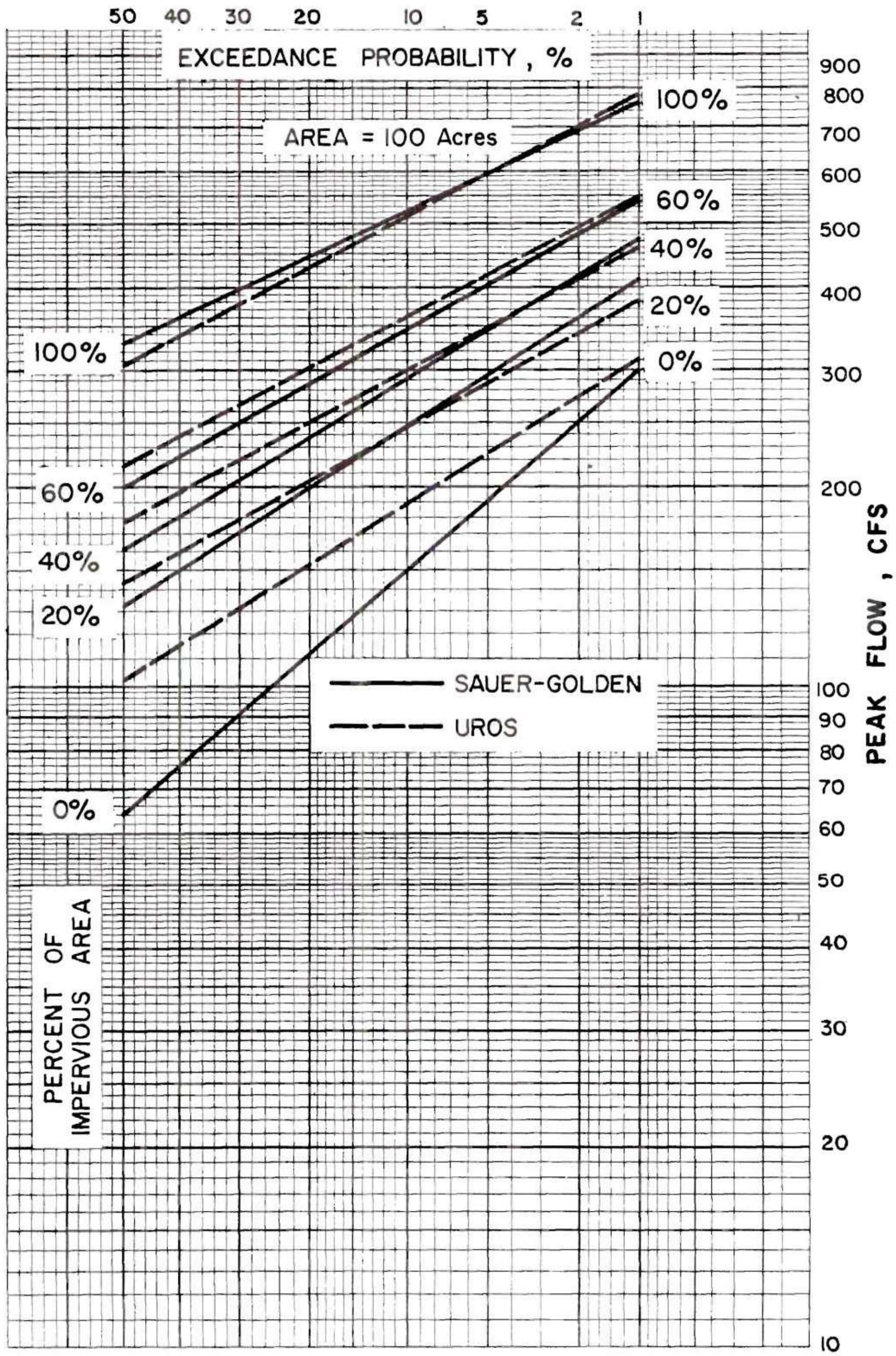

Figure 33. Comparison of Frequency - Peak Discharge Relationships for 100-acre Watersheds.

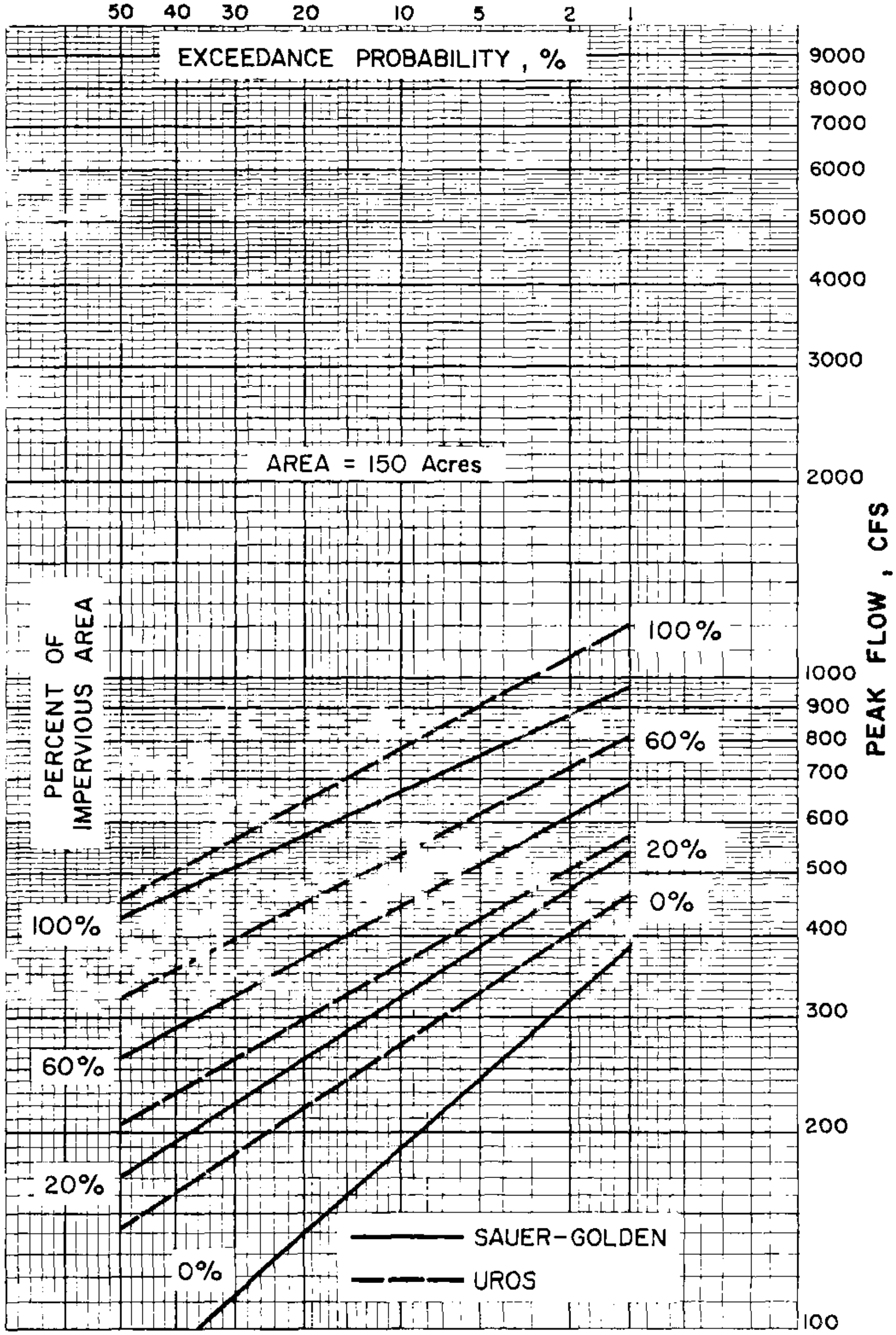

Figure 34. Comparison of Frequency - Peak Discharge Relationships for 150-acre Watersheds.

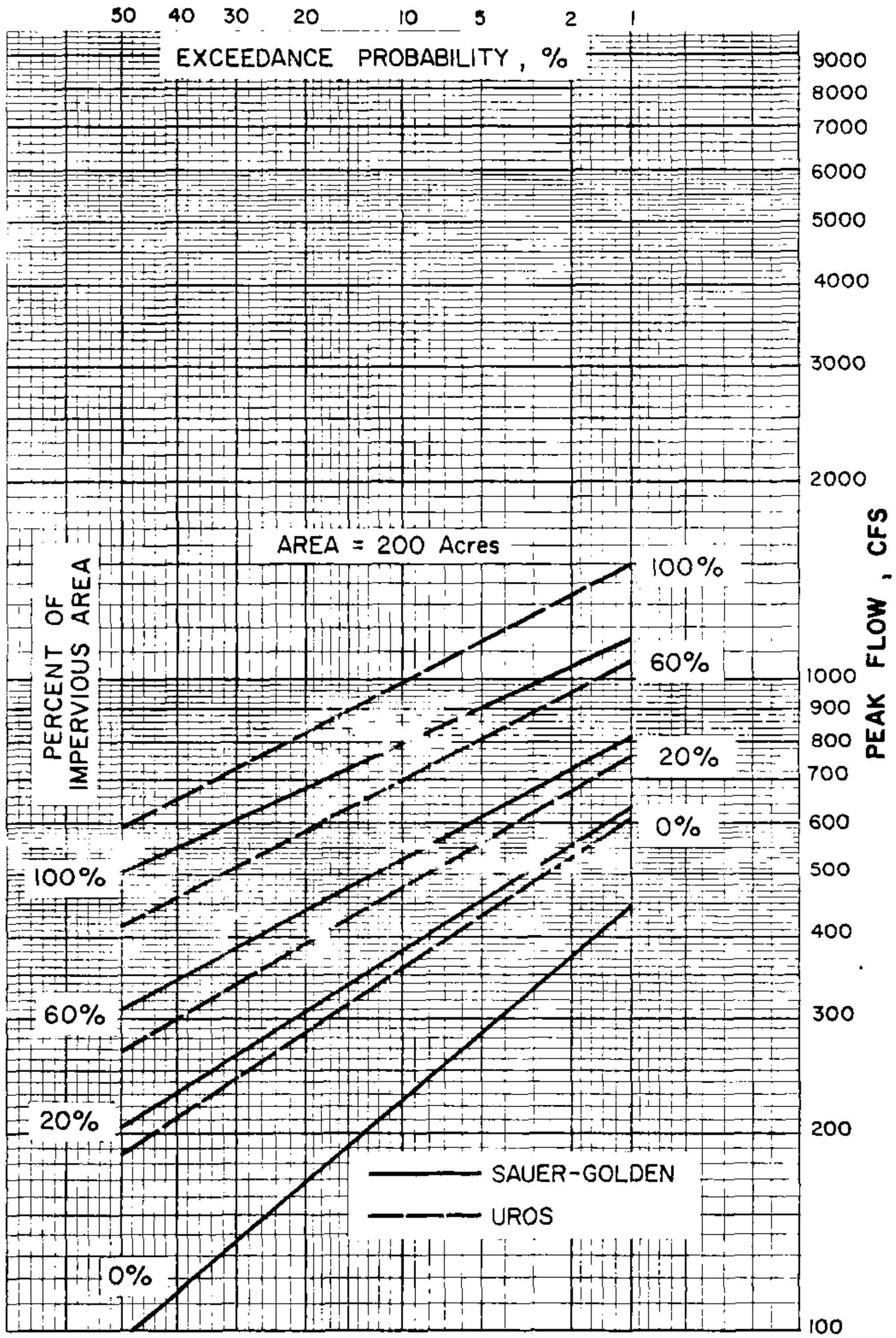

Figure 35. Comparison of Frequency - Peak Discharge Relationships for 200-acre Watersheds.

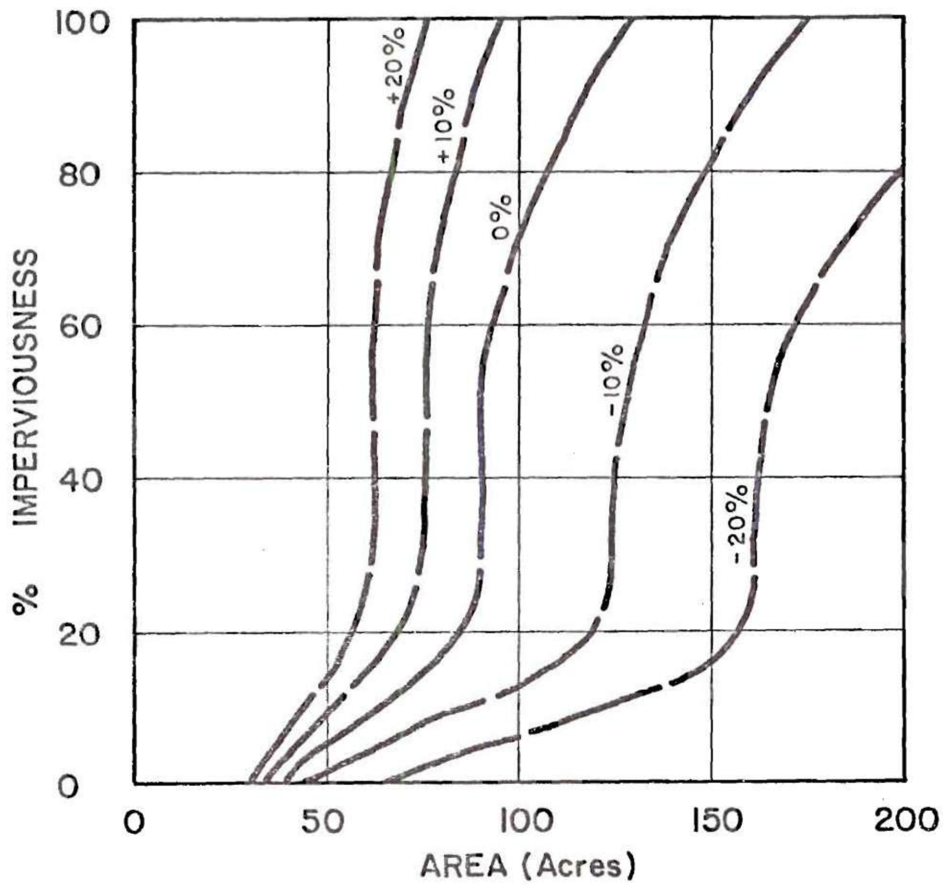

Figure 36. Percent Variation in Estimates of Expected 2-year Return Period Peak Flows as a Function of Watershed Area and Percent Imperiousness.

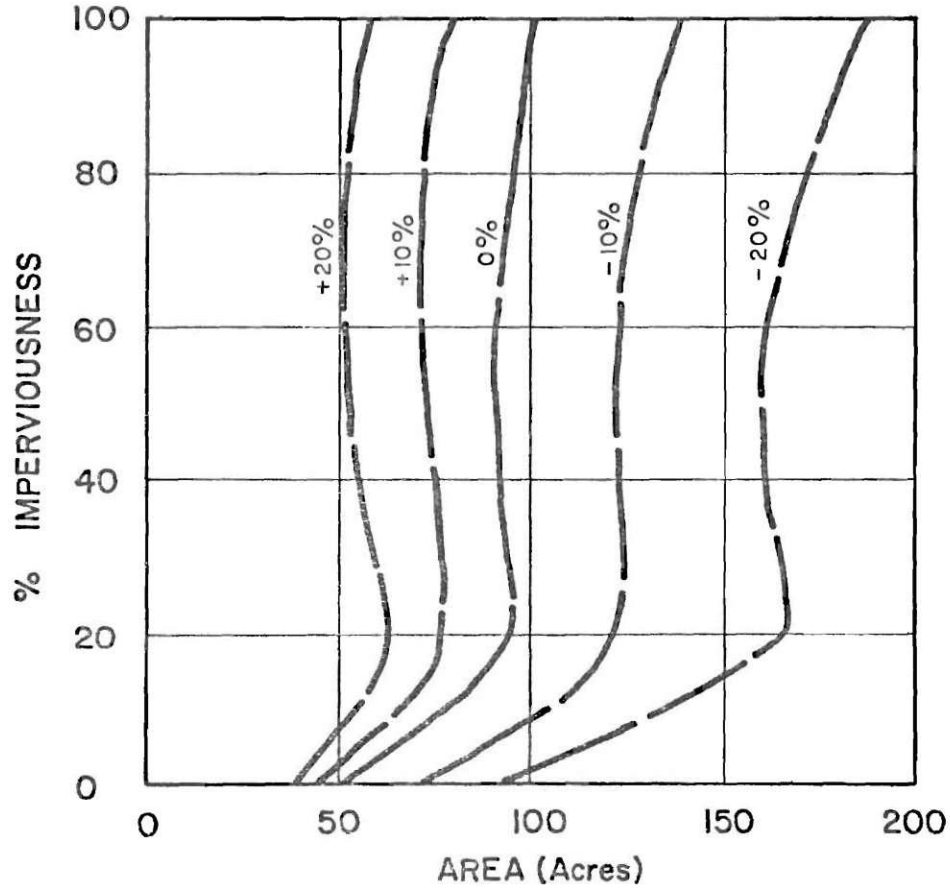

Figure 37. Percent Variation in Estimates of Expected 10-year Return Period Peak Flows as a Function of Watershed Area and Percent Imperiousness.

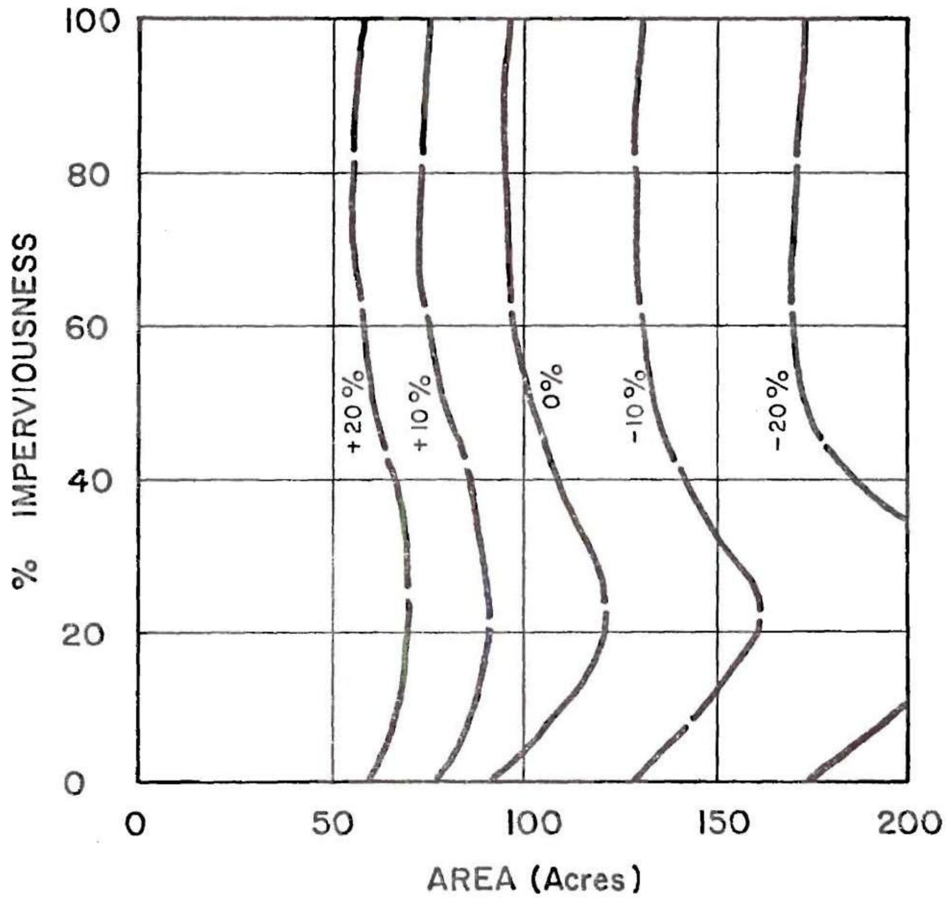

Figure 38. Percent Variation in Estimates of Expected 100-year Return Period Peak Flows as a Function of Watershed Area and Percent Imperiousness.

Ratio of Expected Peak Flows to Expected Mean Annual Peak Flow for Completely Impervious and Fully Sewered Watersheds

Sauer proposed that the ratios of expected peak flood flows to the expected mean annual peak flow ($Q_T:\bar{Q}$) for completely developed (100 percent impervious and fully sewered) watersheds were equal to the ratios of expected peak rainfall intensities to the expected mean annual peak rainfall intensity ($I_T:\bar{I}$) (Sauer, 1974). This assumption is based on a similar one made earlier by Anderson for his work in northern Virginia (Anderson, 1970). To verify this assumption for small watersheds in DeKalb County, flood flow ratios were prepared for completely developed watershed conditions assuming 100 percent impervious cover, 100 percent of the channel length hydraulically modified, and a road density of 10 miles per square mile. For each return period of interest, a ratio of peak flood flow to mean annual peak flow was determined using Equations 4-10 (Table 16). Rainfall intensities were determined from UROS Runoff File - Soil 4 which contains, 1, 5, and 15 minute increments of rainfall rather than runoff. For each of 24 selected rainfall durations, an annual series of peak rainfall intensities was developed for the 25 years of record on the Runoff File. Using an extreme value Type 1, or Gumbel, probability density function, expected peak rainfall intensities were associated with selected return periods for each of the 24 durations (Appendix D). The rainfall intensities were divided by the mean annual peak rainfall intensity to establish ratios for each duration (Appendix E). Averaging these 24 ratios of each return period produced those given in Table 16.

Comparison of flood flow ratios and rainfall intensity ratios in Table 16 appears to state that the ratios of flood flows are slightly greater

Table 16. Comparison of Flood Flow Ratios and Rainfall Intensity Ratios

	(1)	(2)	(3)
$\overline{Q}$	1.00	1.00	1.00
Q_5	1.31	1.27	1.32
Q_{10}	1.55	1.49	1.58
Q_{25}	1.86	1.76	1.91
Q_{50}	2.10	1.96	2.15
Q_{100}	2.33	2.17	2.39

(1) Average Ratio of Urban Flood Flows to Mean Annual Peak Flow for Completely Impervious and Fully Sewered Watershed.

(2) Average Ratio of Rainfall Intensity to Mean Annual Peak Rainfall Intensity from UROS Runoff File for Soil 4.

(3) Average Ratio of Rainfall Intensity to Mean Annual Peak Rainfall Intensity from UROS Runoff File for Soil 4 for Durations of One Hour and Less.

than the ratios of rainfall intensity. Further inspection of the rainfall intensity ratios reveals that they tend to decrease with increasing duration. Accordingly, the short-duration ratios are higher than the average. Since the short-duration-high-intensity rainfalls are the ones most likely creating the highest peak flows on small watersheds, another evaluation of rainfall intensities was made. The ratios for rainfall durations of one hour and less were averaged and are also shown in Table 16. Comparison of these ratios with the flood flow ratios verifies Sauer's assumption with respect to small watersheds in DeKalb County.

CHAPTER IV

CONCLUSIONS

1. For small watersheds (5 to 200 acres) in DeKalb County, Georgia, regression equations developed in this study can be used to estimate, with very good accuracy, the frequency-peak flow relationship obtainable from simulations with UROS: Urban/Rural Flood Simulation Model. Two semi-graphical methods have been developed for solving the regression equations. This conclusion applies to watersheds in which no storage segments are simulated.
2. Simulated peak flood flows from small watersheds (5 to 200 acres) depend primarily on watershed area and percentage of watershed with impervious cover, and to a much smaller degree on improvements (guttering and channel paving) in the channel systems.
3. In the absence of a good record of gaged data on small watersheds (5 to 200 acres) in DeKalb County, Georgia, the expected mean annual peak flow for natural watersheds may be estimated by the equation: $\overline{Q} = 1.32\ A^{0.949}$, where A is the drainage area in acres.
4. Flood flows from a watershed simulated using UROS are dependent on the number and character of source area segments and channel segments used to describe the watershed.
5. The mean annual peak flow from small urbanized watersheds (5 to 200 acres) increases with the imperviousness of the watershed; but, the rate of increase is diminished slightly by the effects of channel storage which usually increases with drainage area. By comparison,

the effects of channelization on expected peak flows are considerably less than the effects of imperviousness.

6. For small watersheds in DeKalb County, Georgia, Leopold's chart for the effects of urbanization on expected peak flows (Leopold, 1968) is not strictly applicable and tends to overestimate the effects.

7. For small watersheds in DeKalb County, Georgia, Stankowski's table for the effects of urbanization on expected peak flows (Stankowski, 1974) is not strictly applicable.

8. The Sauer-Golden method (Golden, 1977) and the regression equations presented in this investigation are not generally compatible for estimating frequency-peak flow relationships for small watersheds (5 to 200 acres) in DeKalb County, Georgia.

9. For small completely impervious and fully sewered watersheds in DeKalb County, Georgia, Sauer's assumption that the ratios of expected peak flows to the expected mean annual peak flow ($Q_T:\bar{Q}$) are equal to the ratios of expected rainfall intensity to expected mean annual peak rainfall intensity ($I_T:\bar{T}$) has been verified with simulated flow data.

APPENDICES

APPENDIX A

Table A. Simulated Data Using Hypothetical Watershed.

CODE	A	I	HM	RD	M	SD
011	5.	0.	0.	0.	5.8	3.6
012	13.	0.	0.	0.	14.9	9.3
013	26.	0.	0.	0.	29.4	18.3
014	39.	0.	0.	0.	43.2	26.9
015	49.	0.	0.	0.	54.5	33.9
016	101.	0.	0.	0.	110.0	68.4
017	208.	0.	0.	0.	214.0	134.7
022	13.	0.	1.	0.	15.4	9.7
023	26.	0.	.5	0.	29.8	18.6
024	39.	0.	.33	0.	43.8	27.3
025	49.	0.	.33	0.	55.1	34.4
026	101.	0.	.29	0.	111.1	69.2
027	208.	0.	.25	0.	215.8	135.9
033	26.	0.	1.	0.	30.6	19.3
034	39.	0.	1.	0.	45.7	28.8
035	49.	0.	1.	0.	56.8	35.9
036	101.	0.	.86	0.	113.9	71.7
037	208.	0.	.75	0.	218.1	138.9
046	101.	0.	1.	0.	116.3	73.7
047	208.	0.	1.	0.	234.0	150.4
051	5.	0.	1.	14.55	6.8	4.0
052	13.	0.	0.	11.19	17.1	9.9
053	26.	0.	0.	5.59	31.4	18.8
054	39.	0.	0.	3.73	45.2	27.4
055	49.	0.	0.	2.97	56.6	34.5
056	101.	0.	0.	2.88	113.9	69.4
057	208.	0.	0.	2.80	220.7	136.5
062	13.	0.	1.	11.19	18.0	10.5
063	26.	0.	.5	5.59	31.9	19.2
064	39.	0.	.33	3.73	45.7	27.7
065	49.	0.	.33	2.97	57.1	34.9
066	101.	0.	.29	2.88	114.9	70.1
067	208.	0.	.25	2.80	222.2	137.5
073	26.	0.	1.	5.59	33.0	20.0
074	39.	0.	1.	3.73	48.0	29.4
075	49.	0.	1.	2.97	58.9	36.4
076	101.	0.	.86	2.88	117.8	72.7
077	208.	0.	.75	2.80	224.3	140.3
086	101.	0.	1.	2.88	120.6	74.9
087	208.	0.	1.	2.80	241.8	152.6
093	26.	0.	0.	11.19	33.4	19.4
094	39.	0.	0.	7.46	47.2	27.9
095	49.	0.	0.	5.94	58.6	35.0

Table A. (Continued)

CODE	A	I	HM	RD	M	SD
096	101.	0.	0.	5.76	117.6	70.3
097	208.	0.	0.	5.59	226.9	138.0
103	26.	0.	.5	11.19	34.3	19.9
104	39.	0.	.33	7.46	47.9	28.4
105	49.	0.	.33	5.94	59.3	35.6
106	101.	0.	.29	5.76	118.9	71.3
107	208.	0.	.25	5.59	228.6	139.1
113	26.	0.	1.	11.19	35.6	20.9
114	39.	0.	1.	7.46	50.5	30.3
115	49.	0.	1.	5.94	61.2	37.2
116	101.	0.	.86	5.76	122.0	73.9
117	208.	0.	.75	5.59	230.3	141.8
126	101.	0.	1.	5.76	125.2	76.3
127	208.	0.	1.	5.59	250.0	155.4
134	39.	0.	0.	11.19	48.9	28.3
135	49.	0.	0.	11.87	61.6	35.7
136	101.	0.	0.	11.52	123.6	71.5
137	208.	0.	0.	11.19	236.4	139.4
144	39.	0.	.33	11.19	49.8	28.9
145	49.	0.	.33	11.87	62.6	36.4
146	101.	0.	.29	11.52	125.3	72.8
147	208.	0.	.25	11.19	238.7	140.8
154	39.	0.	1.	11.19	52.9	31.2
155	49.	0.	1.	11.87	65.7	38.8
156	101.	0.	.86	11.52	130.2	76.7
157	208.	0.	.75	11.19	242.2	144.9
166	101.	0.	1.	11.52	134.1	79.6
167	208.	0.	1.	11.19	266.4	161.4
171	5.	.2	0.	0.	7.4	4.0
172	13.	.2	0.	0.	19.0	10.1
173	26.	.2	0.	0.	37.2	19.8
174	39.	.2	0.	0.	54.6	29.0
175	49.	.2	0.	0.	68.9	36.6
176	101.	.2	0.	0.	138.6	73.2
177	208.	.2	0.	0.	269.3	142.4
182	13.	.2	1.	0.	19.9	10.7
183	26.	.2	.5	0.	38.0	20.3
184	39.	.2	.33	0.	55.4	29.6
185	49.	.2	.33	0.	69.8	37.3
186	101.	.2	.29	0.	140.2	74.5
187	208.	.2	.25	0.	271.7	144.0
193	26.	.2	1.	0.	39.5	21.2
194	39.	.2	1.	0.	58.7	31.7

Table A. (Continued)

CODE	A	I	HM	RD	M	SD
195	49.	.2	1.	0.	72.7	39.4
196	101.	.2	.86	0.	144.8	78.3
197	208.	.2	.75	0.	275.7	147.9
206	101.	.2	1.	0.	148.6	81.0
207	208.	.2	1.	0.	298.7	164.3
211	5.	.2	0.	14.55	8.6	4.3
212	13.	.2	0.	11.19	21.5	10.8
213	26.	.2	0.	5.59	39.6	20.4
214	39.	.2	0.	3.73	56.9	29.7
215	49.	.2	0.	2.97	71.4	37.4
216	101.	.2	0.	2.88	143.1	74.9
217	208.	.2	0.	2.80	276.6	144.3
222	13.	.2	1.	11.19	22.7	11.3
223	26.	.2	.5	5.59	40.5	20.9
224	39.	.2	.33	3.73	57.8	30.2
225	49.	.2	.33	2.97	72.3	38.0
226	101.	.2	.29	2.88	144.7	76.0
227	208.	.2	.25	2.80	278.8	145.6
233	26.	.2	1.	5.59	42.2	21.9
234	39.	.2	1.	3.73	61.5	32.4
235	49.	.2	1.	2.97	75.3	40.1
236	101.	.2	.86	2.88	149.4	79.7
237	208.	.2	.75	2.80	282.5	149.4
246	101.	.2	1.	2.88	153.8	82.5
247	208.	.2	1.	2.80	308.1	167.4
253	26.	.2	0.	11.19	41.9	21.0
254	39.	.2	0.	7.46	59.2	30.2
255	49.	.2	0.	5.94	73.8	38.0
256	101.	.2	0.	5.76	147.3	76.2
257	208.	.2	0.	5.59	283.1	145.8
263	26.	.2	.5	11.19	43.1	21.6
264	39.	.2	.33	7.46	60.2	30.8
265	49.	.2	.33	5.94	74.9	38.7
266	101.	.2	.29	5.76	149.2	77.4
267	208.	.2	.25	5.59	285.7	147.2
273	26.	.2	1.	11.19	45.0	22.6
274	39.	.2	1.	7.46	64.2	33.1
275	49.	.2	1.	5.94	77.9	40.7
276	101.	.2	.86	5.76	154.1	80.9
277	208.	.2	.75	5.59	289.4	151.0
286	101.	.2	1.	5.76	159.1	83.9
287	208.	.2	1.	5.59	317.4	170.7
293	26.	.4	1.	5.59	52.5	23.9

Table A. (Continued)

CODE	A	I	HM	RD	M	SD
294	39.	.2	0.	11.19	61.1	30.5
295	49.	.2	0.	11.87	77.3	38.6
296	101.	.2	0.	11.52	154.0	77.0
297	208.	.2	0.	11.19	293.1	147.6
304	39.	.2	.33	11.19	62.3	31.3
305	49.	.2	.33	11.87	78.7	39.4
306	101.	.2	.29	11.52	156.3	78.5
307	208.	.2	.25	11.19	296.4	149.8
314	39.	.2	1.	11.19	66.9	33.9
315	49.	.2	1.	11.87	82.9	42.1
316	101.	.2	.86	11.52	163.3	83.4
317	208.	.2	.75	11.19	303.2	154.9
326	101.	.2	1.	11.52	169.1	86.6
327	208.	.2	1.	11.19	336.4	176.3
331	5.	.4	0.	0.	9.3	4.4
332	13.	.4	0.	0.	23.8	11.2
333	26.	.4	0.	0.	46.4	21.8
334	39.	.4	0.	0.	67.9	31.9
335	49.	.4	0.	0.	86.0	40.4
336	101.	.4	0.	0.	171.9	80.9
337	208.	.4	0.	0.	331.5	156.6
342	13.	.4	1.	0.	24.9	11.7
343	26.	.4	.5	0.	47.5	22.5
344	39.	.4	.33	0.	69.1	32.6
345	49.	.4	.33	0.	87.3	41.2
346	101.	.4	.29	0.	174.2	82.2
347	208.	.4	.25	0.	334.7	158.2
353	26.	.4	1.	0.	49.5	23.4
354	39.	.4	1.	0.	73.7	35.1
355	49.	.4	1.	0.	91.2	43.7
356	101.	.4	.86	0.	180.8	85.6
357	208.	.4	.75	0.	341.5	163.5
366	101.	.4	1.	0.	186.4	89.7
367	208.	.4	1.	0.	373.8	182.5
371	5.	.4	0.	14.55	10.8	4.6
372	13.	.4	0.	11.19	26.6	11.8
373	26.	.4	0.	5.59	49.2	22.5
374	39.	.4	0.	3.73	70.6	32.6
375	49.	.4	0.	2.97	88.8	41.2
376	101.	.4	0.	2.88	177.3	82.2
377	208.	.4	0.	2.80	340.3	158.2
382	13.	.4	1.	11.19	28.1	12.2
383	26.	.4	.5	5.59	50.4	23.1

Table A. (Continued)

CODE	A	I	HM	RD	M	SD
384	39.	.4	.33	3.73	71.8	33.2
385	49.	.4	.33	2.97	90.1	41.9
386	101.	.4	.29	2.88	179.6	83.4
387	208.	.4	.25	2.80	343.6	159.9
394	39.	.4	1.	3.73	76.7	35.6
395	49.	.4	1.	2.97	94.2	44.2
396	101.	.4	.86	2.88	186.4	87.7
397	208.	.4	.75	2.80	350.4	165.0
406	101.	.4	1.	2.88	192.4	90.9
407	208.	.4	1.	2.80	385.3	185.0
413	26.	.4	0.	11.19	51.8	23.1
414	39.	.4	0.	7.46	73.2	33.3
415	49.	.4	0.	5.94	91.5	41.9
416	101.	.4	0.	5.76	182.2	83.5
417	208.	.4	0.	5.59	348.8	159.9
423	26.	.4	.5	11.19	53.3	23.7
424	39.	.4	.33	7.46	74.5	33.9
425	49.	.4	.33	5.94	93.0	42.7
426	101.	.4	.29	5.76	184.7	84.8
427	208.	.4	.25	5.59	352.3	161.7
433	26.	.4	1.	11.19	55.8	24.4
434	39.	.4	1.	7.46	79.8	36.1
435	49.	.4	1.	5.94	97.1	44.6
436	101.	.4	.86	5.76	191.8	89.0
437	208.	.4	.75	5.59	359.4	166.8
446	101.	.4	1.	5.76	198.3	92.0
447	208.	.4	1.	5.59	396.4	187.4
454	39.	.4	0.	11.19	75.4	33.6
455	49.	.4	0.	11.87	95.6	42.6
456	101.	.4	0.	11.52	189.7	84.6
457	208.	.4	0.	11.19	360.9	161.2
464	39.	.4	.33	11.19	76.9	34.4
465	49.	.4	.33	11.87	97.3	43.4
466	101.	.4	.29	11.52	192.8	86.4
467	208.	.4	.25	11.19	365.5	163.5
474	39.	.4	1.	11.19	83.0	36.6
475	49.	.4	1.	11.87	102.7	45.5
476	101.	.4	.86	11.52	202.1	91.3
477	208.	.4	.75	11.19	376.7	170.7
486	101.	.4	1.	11.52	209.6	94.1
487	208.	.4	1.	11.19	417.9	192.3
491	5.	1.	0.	0.	17.0	7.6
492	13.	1.	0.	0.	42.3	18.9

Table A. (Continued)

CODE	A	I	HM	RD	M	SD
493	26.	1.	0.	0.	82.0	36.8
494	39.	1.	0.	0.	119.1	53.9
495	49.	1.	0.	0.	151.1	67.7
496	101.	1.	0.	0.	299.8	136.2
497	208.	1.	0.	0.	568.5	258.3
502	13.	1.	1.	0.	44.9	19.7
503	26.	1.	.5	0.	84.6	37.8
504	39.	1.	.33	0.	121.9	55.0
505	49.	1.	.33	0.	154.1	69.1
506	101.	1.	.29	0.	305.2	138.6
507	208.	1.	.25	0.	576.1	261.8
513	26.	1.	1.	0.	89.0	39.0
514	39.	1.	1.	0.	132.2	58.2
515	49.	1.	1.	0.	163.3	72.6
516	101.	1.	.86	0.	320.6	145.0
517	208.	1.	.75	0.	593.9	267.2
526	101.	1.	1.	0.	332.8	148.3
527	208.	1.	1.	0.	662.8	298.0
531	5.	1.	0.	14.55	19.9	8.4
532	13.	1.	0.	11.19	47.4	20.0
533	26.	1.	0.	5.59	86.8	38.0
534	39.	1.	0.	3.73	123.9	55.2
535	49.	1.	0.	2.97	156.2	69.4
536	101.	1.	0.	2.88	309.1	139.0
537	208.	1.	0.	2.80	582.7	262.4
542	13.	1.	1.	11.19	51.4	21.4
543	26.	1.	.5	5.59	89.5	38.7
544	39.	1.	.33	3.73	126.5	56.0
545	49.	1.	.33	2.97	159.2	70.5
546	101.	1.	.29	2.88	314.3	141.0
547	208.	1.	.25	2.80	590.2	265.1
553	26.	1.	1.	5.59	94.9	40.6
554	39.	1.	1.	3.73	138.0	59.5
555	49.	1.	1.	2.97	168.8	73.6
556	101.	1.	.86	2.88	330.2	147.0
557	208.	1.	.75	2.80	607.6	269.3
566	101.	1.	1.	2.88	343.6	150.7
567	208.	1.	1.	2.80	681.7	301.6
573	26.	1.	0.	11.19	91.1	38.6
574	39.	1.	0.	7.46	128.1	56.0
575	49.	1.	0.	5.94	160.9	70.5
576	101.	1.	0.	5.76	317.6	141.0
577	208.	1.	0.	5.59	596.0	265.2

Table A. (Concluded)

CODE	A	I	HM	RD	M	SD
583	26.	1.	.5	11.19	94.7	40.0
584	39.	1.	.33	7.46	131.2	[illegible]7.1
585	49.	1.	.33	5.94	164.3	[illegible]1.8
586	101.	1.	.29	5.76	323.3	143.4
587	208.	1.	.25	5.59	604.1	268.1
593	26.	1.	1.	11.19	101.3	42.2
594	39.	1.	1.	7.46	143.8	60.8
595	49.	1.	1.	5.94	174.3	74.9
596	101.	1.	.86	5.76	339.4	148.8
597	208.	1.	.75	5.59	620.7	271.0
606	101.	1.	1.	5.76	354.1	152.8
607	208.	1.	1.	5.59	700.7	305.1
614	39.	1.	0.	11.19	131.5	56.2
615	49.	1.	0.	11.87	167.0	70.5
616	101.	1.	0.	11.52	328.5	141.5
617	208.	1.	0.	11.19	613.7	266.1
624	39.	1.	.33	11.19	135.0	57.6
625	49.	1.	.33	11.87	171.0	72.2
626	101.	1.	.29	11.52	335.4	144.4
627	208.	1.	.25	11.19	623.0	269.7
634	39.	1.	1.	11.19	149.9	62.4
635	49.	1.	1.	11.87	184.6	77.0
636	101.	1.	.86	11.52	356.9	152.0
637	208.	1.	.75	11.19	646.1	273.6
646	101.	1.	1.	11.52	374.4	156.7
647	208.	1.	1.	11.19	735.2	311.4

Code - Reference number to simulated data
A - Drainage area in acres
I - Decimal fraction of impervious area
HM - Decimal fraction of channel length hydraulically modified
RD - Road density in miles per square mile
M - Mean of the simulated annual peak flow series in CFS
SD - Standard deviation of the simulated annual peak flow series in CFS

APPENDIX B*

A SIMPLE METHOD FOR ESTIMATING EXPECTED PEAK STORMWATER FLOWS FOR SMALL WATERSHEDS IN DeKALB COUNTY, GEORGIA

A simple method has been developed for estimating expected peak stormwater flow rates for small watersheds from 5 acres to 200 acres in DeKalb County, Georgia. It is based on simulation of the hydrologic response of a hypothetical watershed using UROS: Urban/Rural Flood Simulation Model. The hypothetical watershed is representative of the physiography of the area. The model, UROS was developed specifically for DeKalb County by the Georgia Institute of Technology during 1974 and 1975.

The simple method involves the semi-graphical solution of regression equations generated from the simulation results. The regression equations estimate the peak flow as a function of return period, drainage area, imperviousness, road density and percent of channel length that has been hydraulically modified. Separate equations for the mean and standard deviation of the annual peak flow series as well as for selected return periods are presented in Table B-1. The charts used in the graphical solution of these equations are given in Figures B-1 through B-9. The proper procedure for using these charts and applying the simple method is outlined below and illustrated by an example.

<u>Discussion of Example</u>:

From available maps and through field verification of ridge lines, the drainage area of the watershed has been estimated as 150 acres. From maps, aerial photographs, and field inspection, the percent of area with

Table B-1. Regression Equations for Estimating Expected Peak Flows

$$\overline{Q} = 1.319(A)^{0.949} \left[(1+I)^{1.657}\right]^{\left[\log(1+I)\right]^{0.1}} (1+RD)^{.044} (1+HM)^{0.116} \quad (4)$$

$$SD = 0.811(A)^{0.956} \left[(1+I)^{2.890}\right]^{\left[\log(1+I)\right]^{0.9}} (1+HM)^{0.124} (1+RD)^{0.020} \quad (5)$$

$$Q_5 = 1.895(A)^{0.951} \left[(1+I)^{1.704}\right]^{\left[\log(1+I)\right]^{0.2}} (1+RD)^{0.037} (1+HM)^{0.119} \quad (6)$$

$$Q_{10} = 2.331(A)^{0.952} \left[(1+I)^{1.824}\right]^{\left[\log(1+I)\right]^{0.3}} (1+HM)^{0.120} (1+RD)^{0.034} \quad (7)$$

$$Q_{25} = 2.992(A)^{0.952} \left[(1+I)^{1.964}\right]^{\left[\log(1+I)\right]^{0.4}} (1+HM)^{0.120} (1+RD)^{0.032} \quad (8)$$

$$Q_{50} = 3.441(A)^{0.953} \left[(1+I)^{2.035}\right]^{\left[\log(1+I)\right]^{0.45}} (1+HM)^{0.121} (1+RD)^{0.030} \quad (9)$$

$$Q_{100} = 3.864(A)^{.953} \left[(1+I)^{2.004}\right]^{\left[\log(1+I)\right]^{0.45}} (1+HM)^{0.121} (1+RD)^{0.029} \quad (10)$$

$\overline{Q}$ = expected mean annual peak flow in CFS
SD = standard deviation of the annual peak flow series in CFS
Q_T = expected peak flow for return period T in CFS
A = drainage area in acres
I = decimal fraction of impervious area
RD = road density in miles per square mile
HM = decimal fraction of channel length hydraulically modified

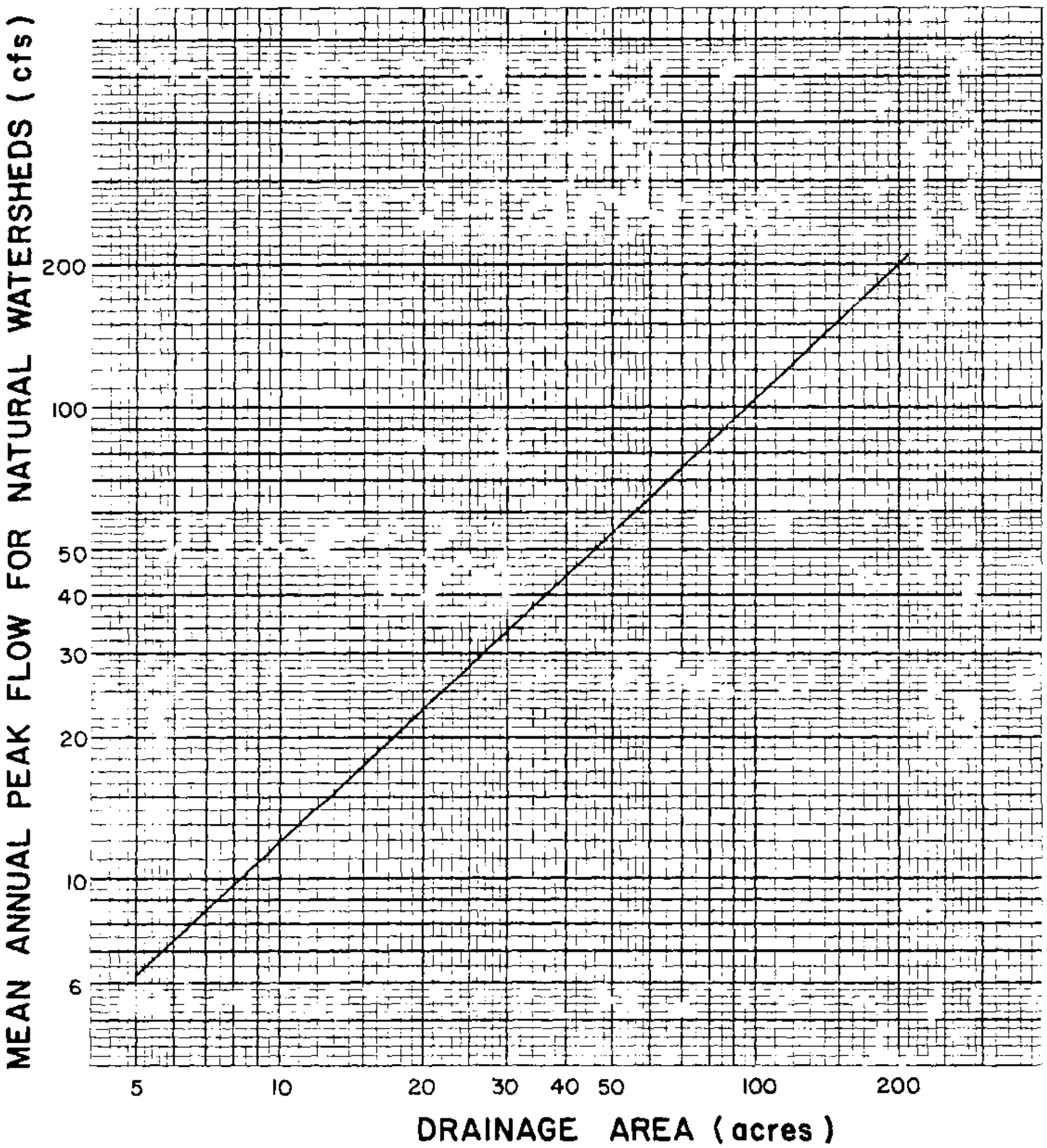

Figure B-1. Mean Annual Peak Flow for Small Natural Watersheds vs. Drainage Area.

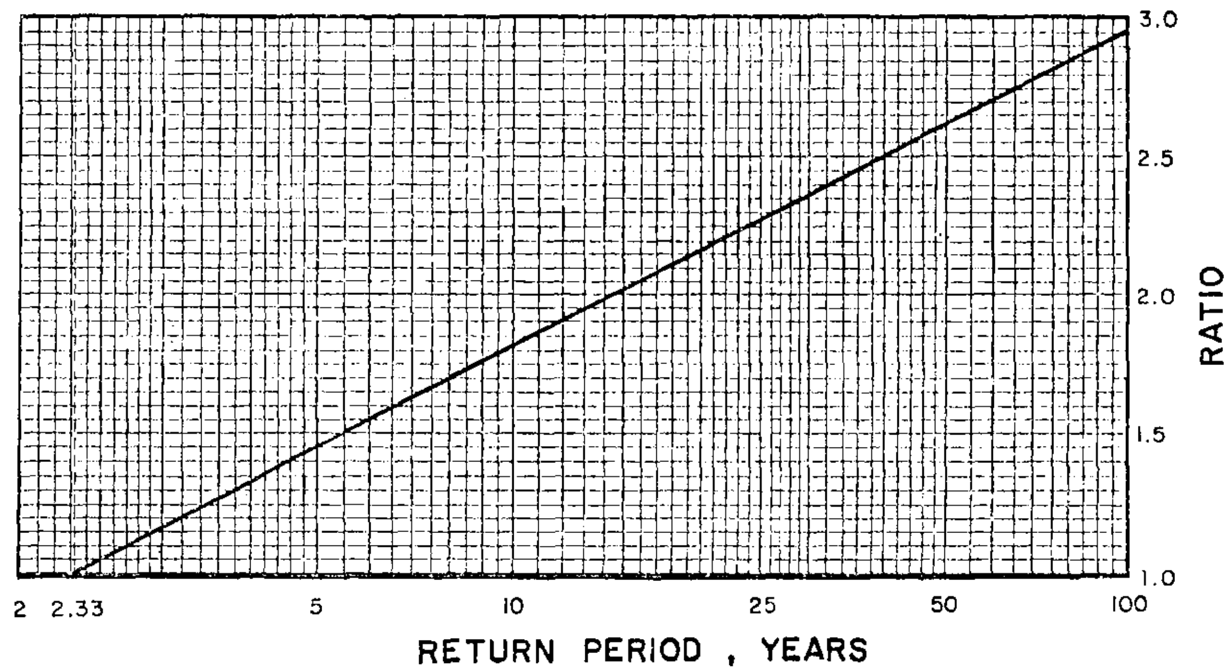

Figure B-2. Ratio of Expected Peak Flow to Mean Annual Peak Flow for Small Natural Watersheds vs. Return Period.

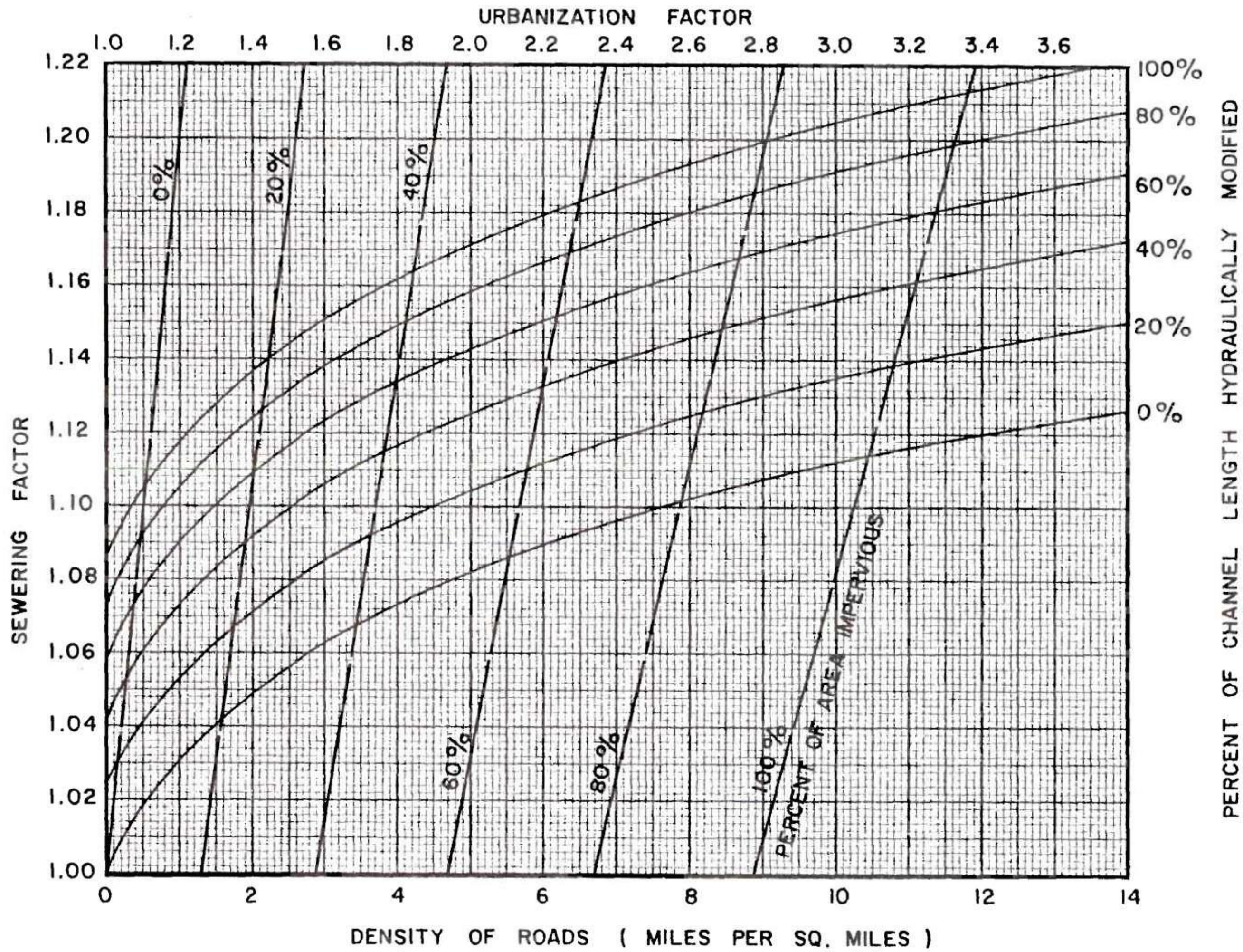

Figure B-3. Urbanization Factor Chart for the Expected Mean of Annual Peak Flow Series.

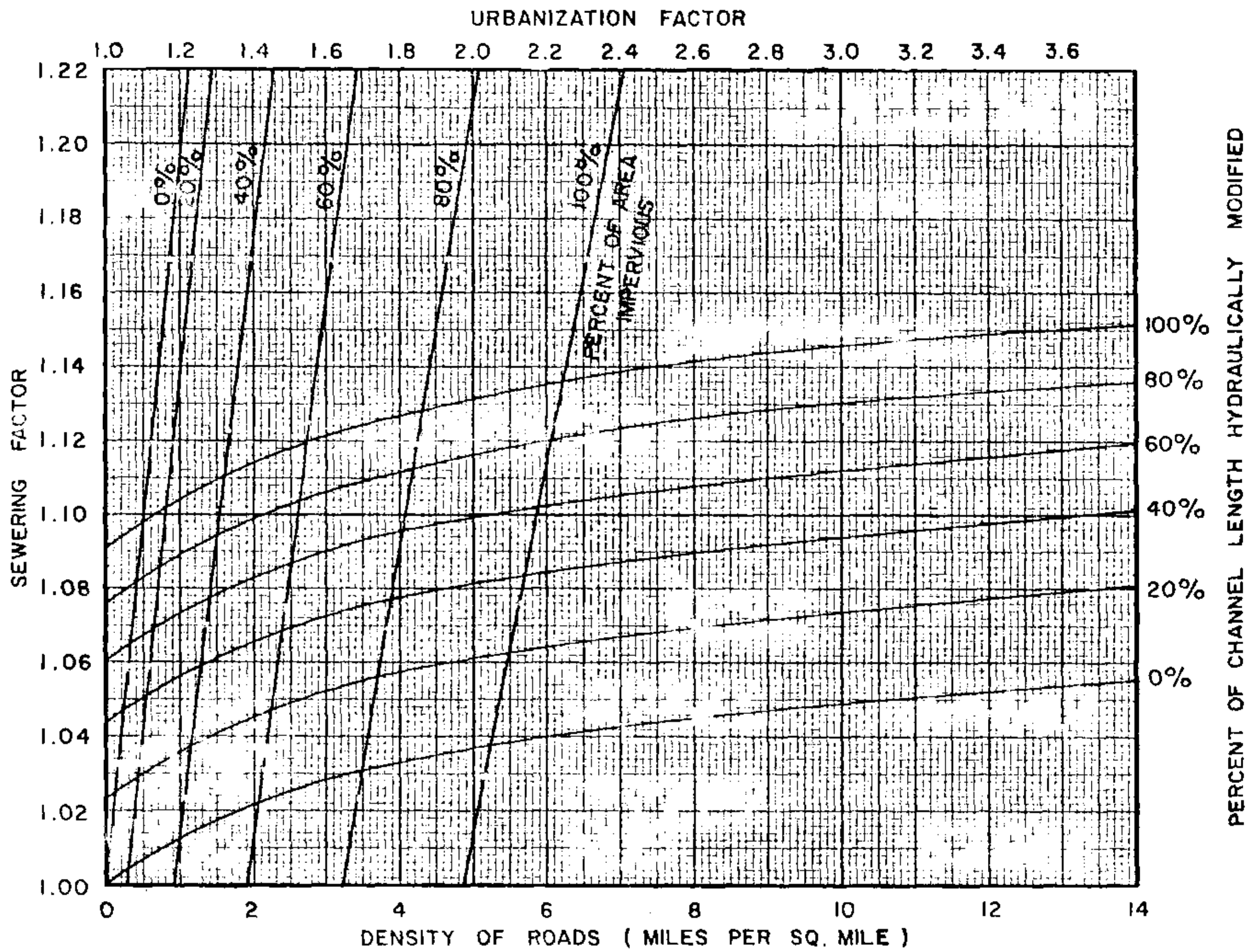

Figure B-4. Urbanization Factor Chart for the Standard Deviation of the Annual Peak Flow Series.

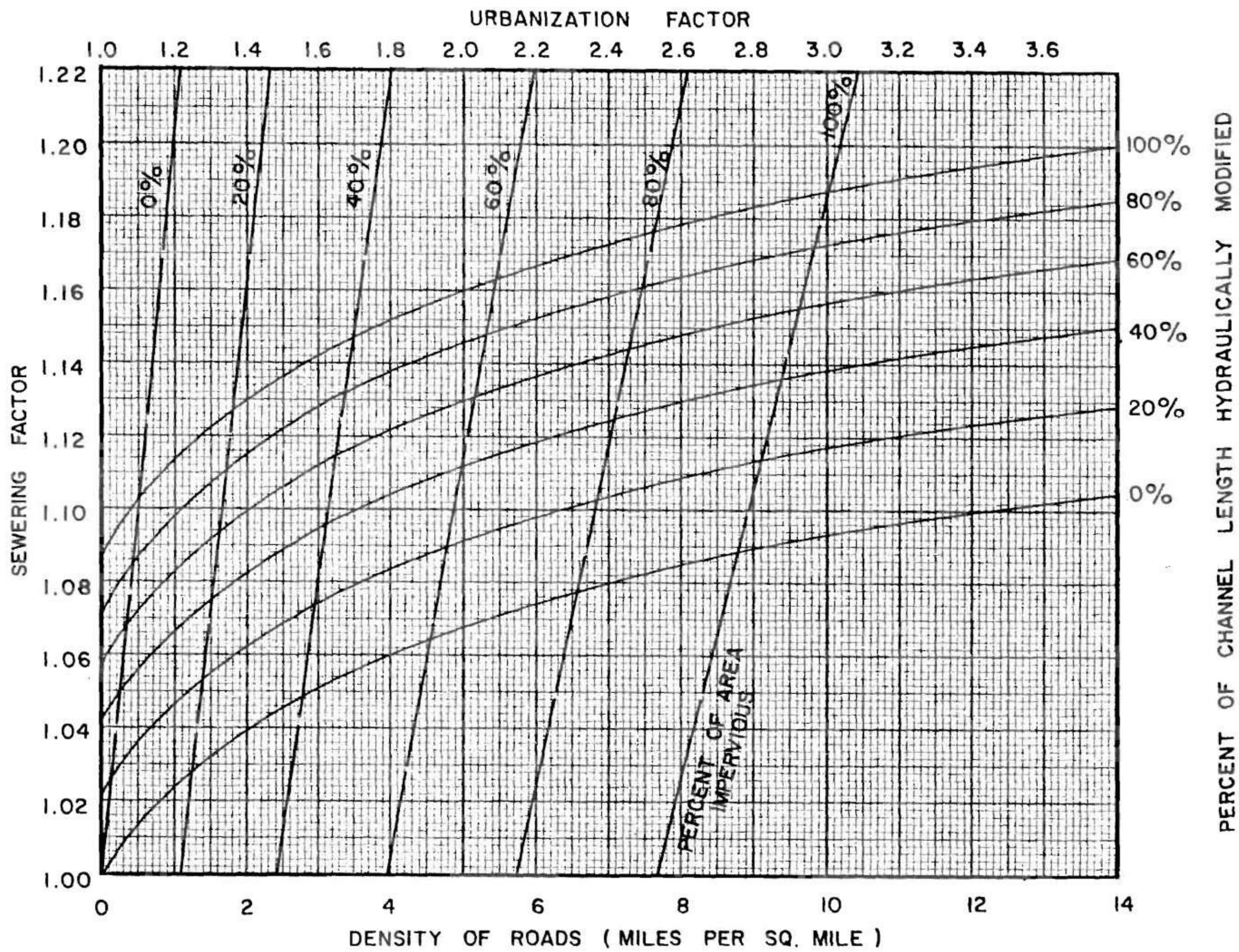

Figure B-5. Urbanization Factor Chart for the Expected 5-year Return Period Peak Flow.

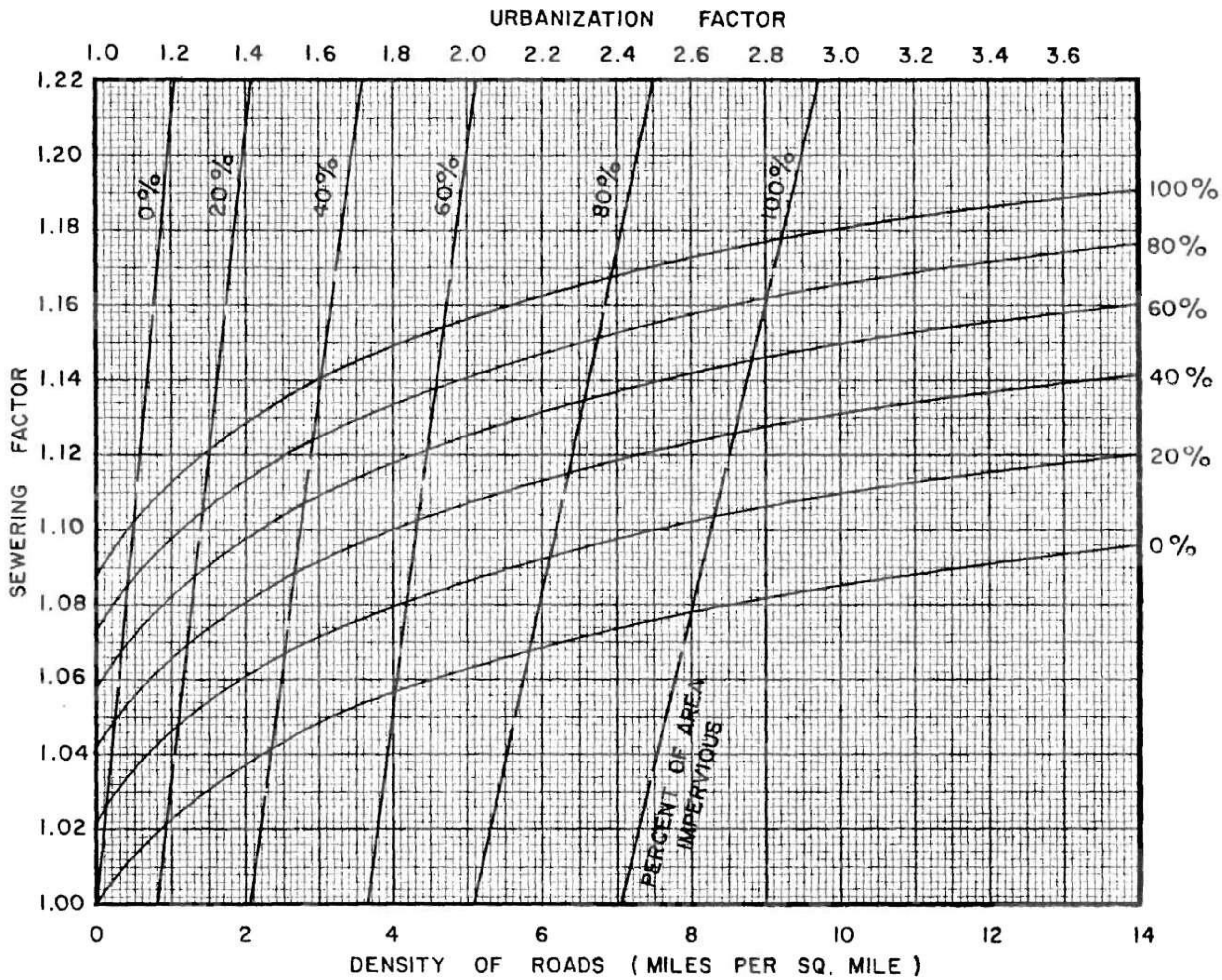

Figure B-6. Urbanization Factor Chart for the Expected 10-year Return Period Peak Flow.

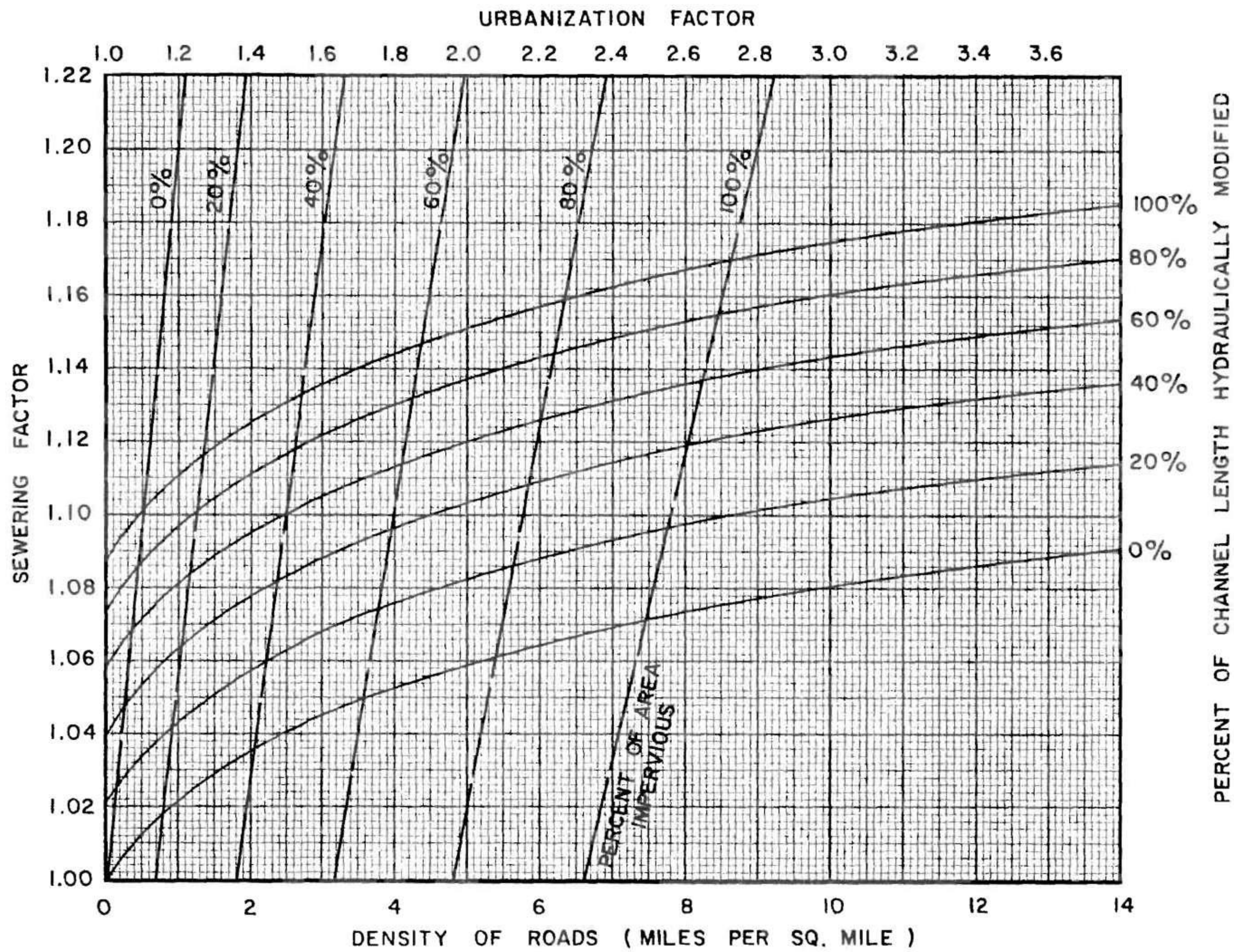

Figure B-7. Urbanization Factor Chart for the Expected 25-year Return Period Peak Flow.

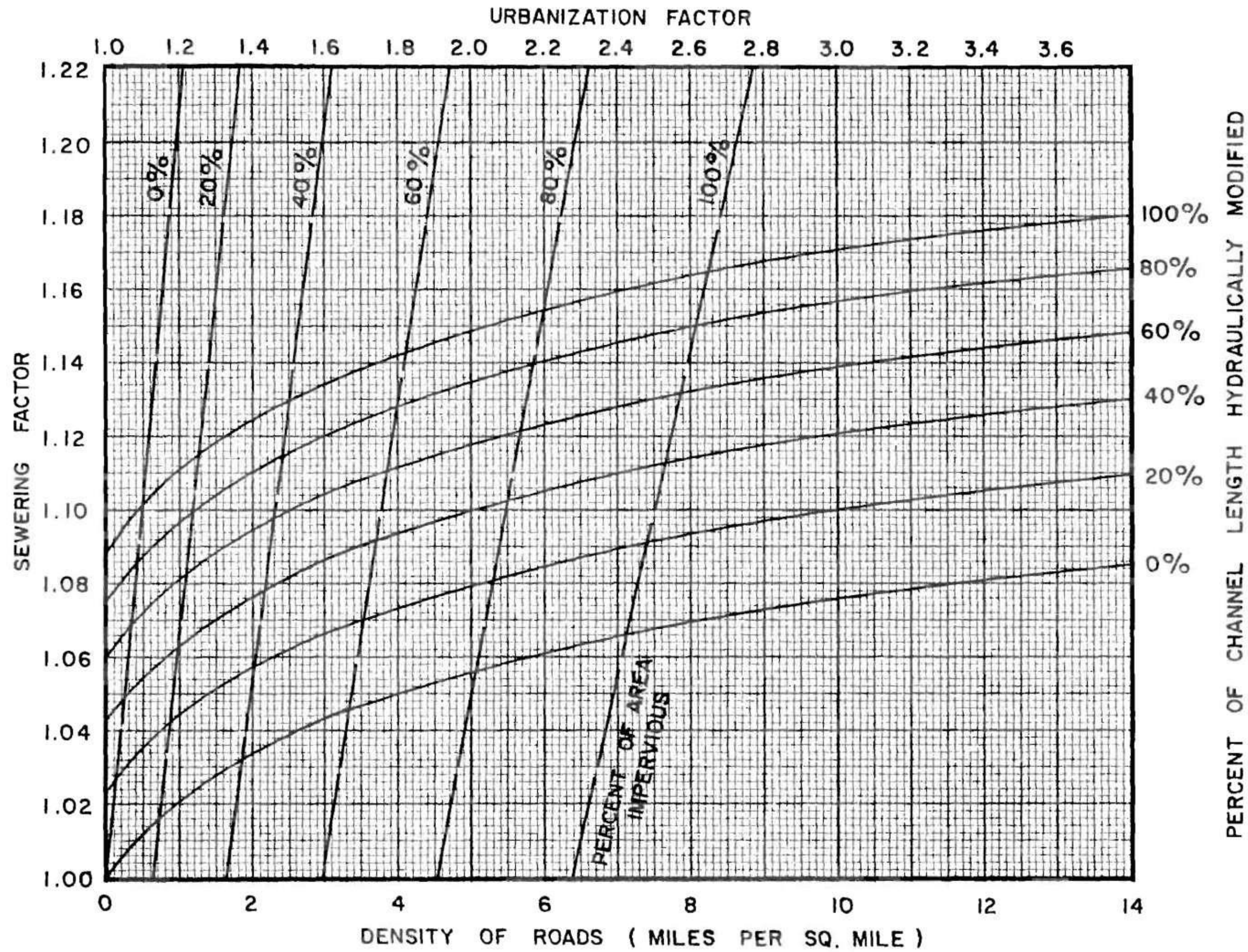

Figure B-8. Urbanization Factor Chart for the Expected 50-year Return Period Peak Flow.

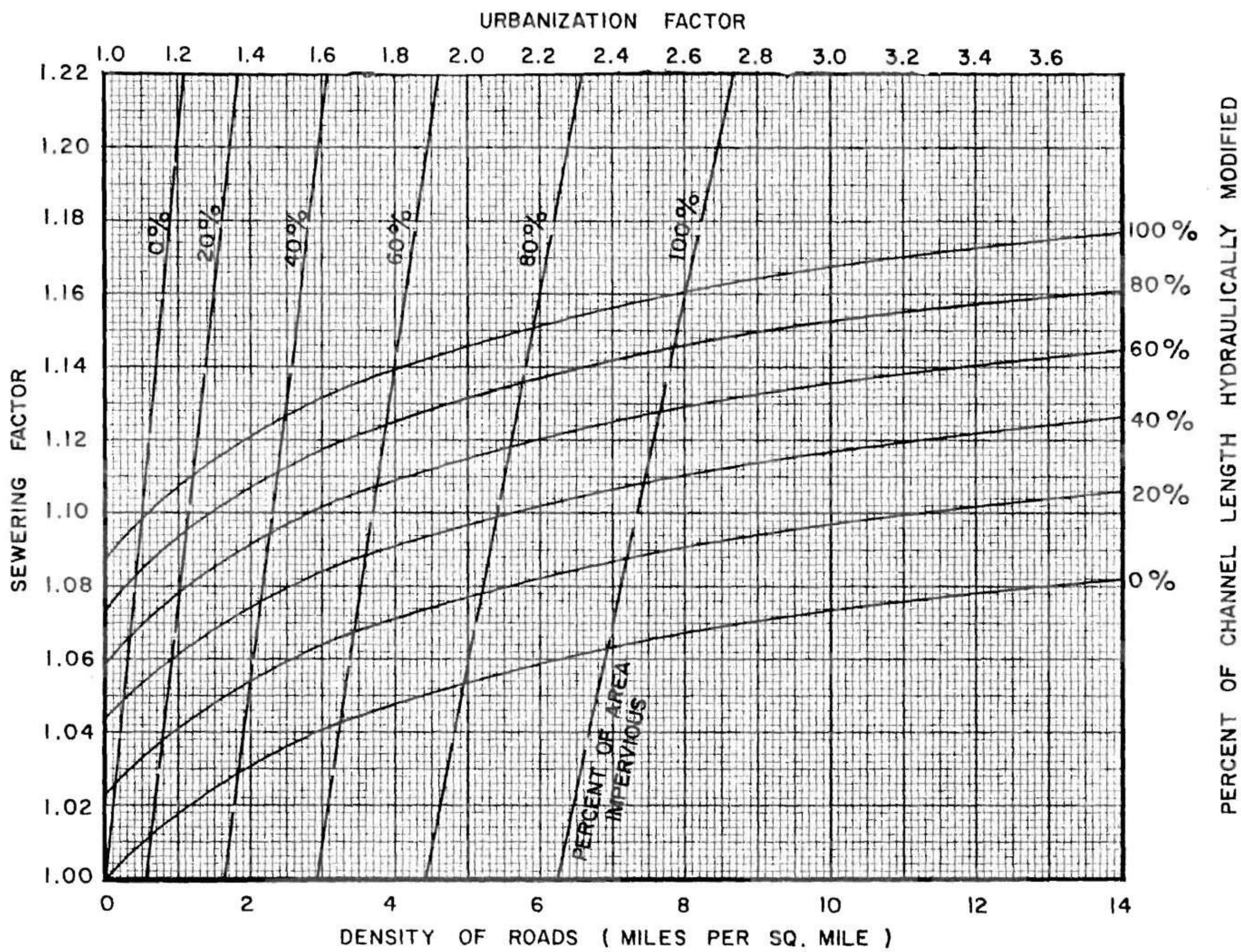

Figure B-9. Urbanization Factor Chart for the Expected 100-year Return Period Peak Flow.

with impervious cover, including streets and roadways, has been estimated as 25 percent. Additionally, it has been determined that the density of roads is 10 miles per square mile and that 20 percent of the natural channel length in the watershed has been hydraulically modified. Based on these characteristics of the urban watershed, an estimate of the frequency-peak discharge relationship can be made. It is also desirable to compare this post-development condition with the frequency-peak discharge relationship for the 150 acre watershed in its natural, predevelopment state.

Summary of Given Data for Example Problem:

A = 150 acres

I = 25% imperviousness

RD = 10 mi./sq/mi. road density

HM = 20% of channel hydraulically modified

Procedures for Applying Simple Method:

Step 1. Given the drainage area, determine the expected mean annual peak flow for a natural watershed using Figure B-1. Example: Figure B-1 yields $\overline{Q}_{(n)}$ = 153 cfs.

Step 2. From Figure B-2, the ratios of expected peak flow to mean annual flow ($Q_t:\overline{Q}$) are determined and multiplied by $\overline{Q}_{(n)}$ to yield the frequency-peak discharge relationship for the watershed in natural conditions.

Example: $\overline{Q}_{(n)}$ = 153 x 1.00 = 153 CFS

$Q_{5_{(n)}}$ = 153 x 1.45 = 222 CFS

$Q_{10_{(n)}}$ = 153 x 1.82 = 278 CFS

$Q_{25_{(n)}}$ = 153 x 2.29 = 350 CFS

$Q_{50_{(n)}}$ = 153 x 2.63 = 407 CFS

$$Q_{100_{(n)}} = 153 \times 2.97 = 454 \text{ CFS}$$

Step 3. Figures B-3 through B-9 are used to determine the effects of urbanization on the expected peak flows. Locate the correct Urbanization Factor Chart for the desired return period. Enter the chart with the given value of road density (RD); move vertically until reaching a drawn of interpolated curve representing the given value of percent of channel length hydraulically modified (HM); reading horizontally to the left yields the Sewering Factor. On this same horizontal line read back right to the drawn or interpolated curve for the percent of impervious area (I) ; read vertically to the scale for the Urbanization Factor. (See Figure B-10 for the example). Repeat Step 3 for each return period taking note to use the proper Urbanization Factor Chart.

Example: For 2.33-year return period, RD = 10 miles per square mile, HM = 20%, and I = 25%, the Urbanization Factor (UF) taken from Figure B-3 is 1.54 (See Figure B-10). Similarly $UF_{(5)} = 1.42$, $UF_{(10)} = 1.37$, $UF_{(25)} = 1.32$, $UF_{(50)} = 1.30$, $UF_{(100)} = 1.28$.

Step 4: The expected peak flows for the watershed in natural conditions, determined in Step 2, are multiplied by the appropriate Urbanization Factors, determined from Step 3, to yield the estimated frequency-peak discharge relationship for the watershed in urbanized conditions. Note that the Sewering Factor is included in the Urbanization Factor. It should not be used as an additional multiplier! If the user wishes to determine the separate effect

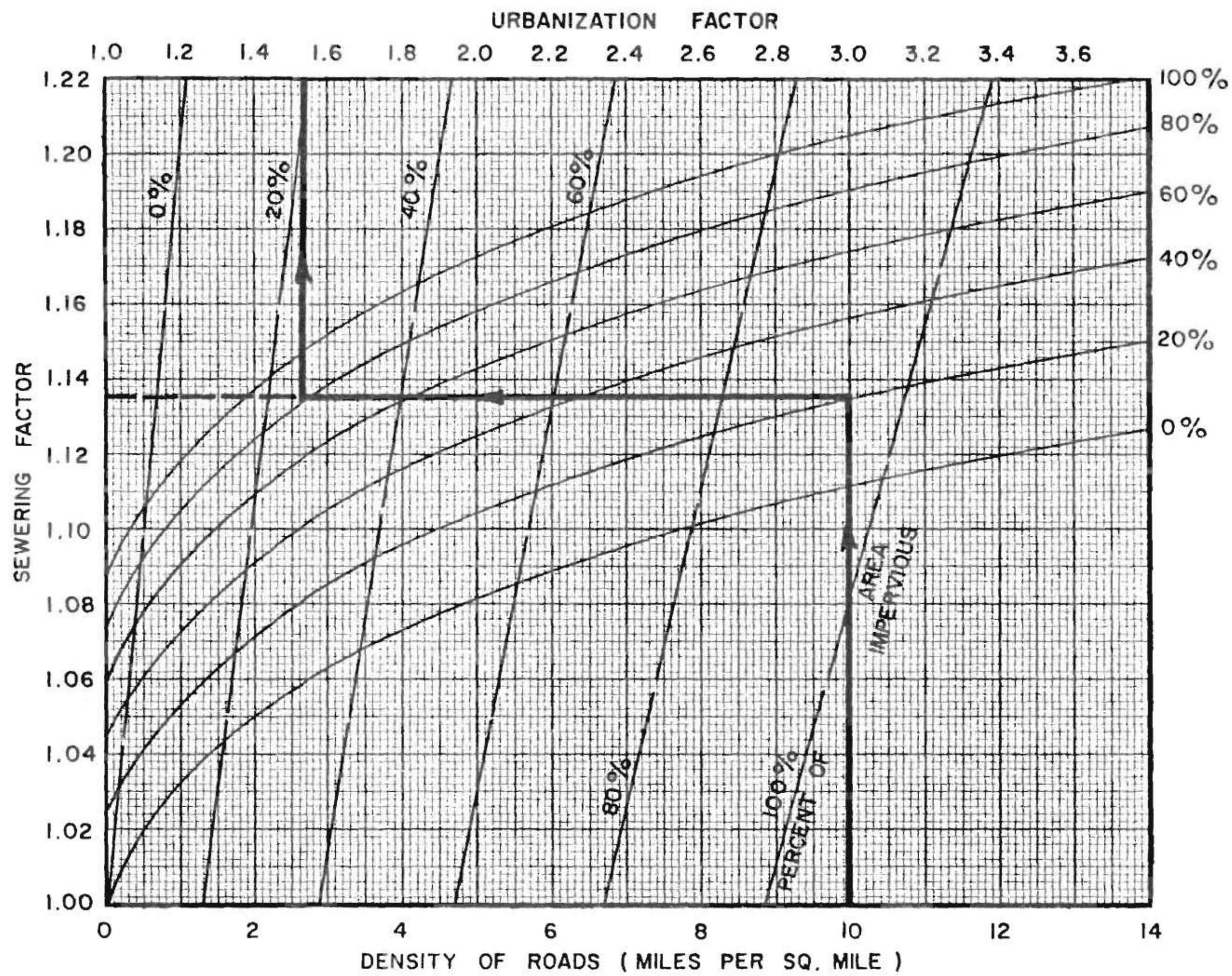

Figure B-10. Example Use of Urbanization Factor Chart.

of impervious area, one need only divide the Urbanization Factor by the Sewering Factor. Having estimated the frequency-peak discharge relationship for urbanized conditions, the estimates should be rounded-off to two significant digits.

Example: $\overline{Q}_{(u)} = 153 \times 1.54 = 236$ (say 240 CFS)

$Q_{5(u)} = 222 \times 1.42 = 315$ (say 320 CFS)

$Q_{10(u)} = 278 \times 1.37 = 381$ (say 380 CFS)

$Q_{25(u)} = 350 \times 1.32 = 462$ (say 460 CFS)

$Q_{50(u)} = 402 \times 1.30 = 523$ (say 520 CFS)

$Q_{100(u)} = 454 \times 1.28 = 581$ (say 580 CFS)

Step 5: The estimated frequency-peak discharge relationships for both natural and urbanized conditions should be plotted to check for errors in calculations and to give a graphical indication of the effects of urbanization on expected peak flows. A plot of the estimated peak flows on arithmetic-extreme value Type I paper will be a straight line since the regression equations for specific return periods (Table B-1) were generated from the extreme value Type I, or Gumble, probability density function. A plot of the estimated peak flows on log-normal probability paper will very closely approximate a straight line.

Example: See Figure B-11 for plots on Gumble paper and Figure B-12 for plots on log-normal probability paper.

Summary: All calculations are summarized in Table B-2.

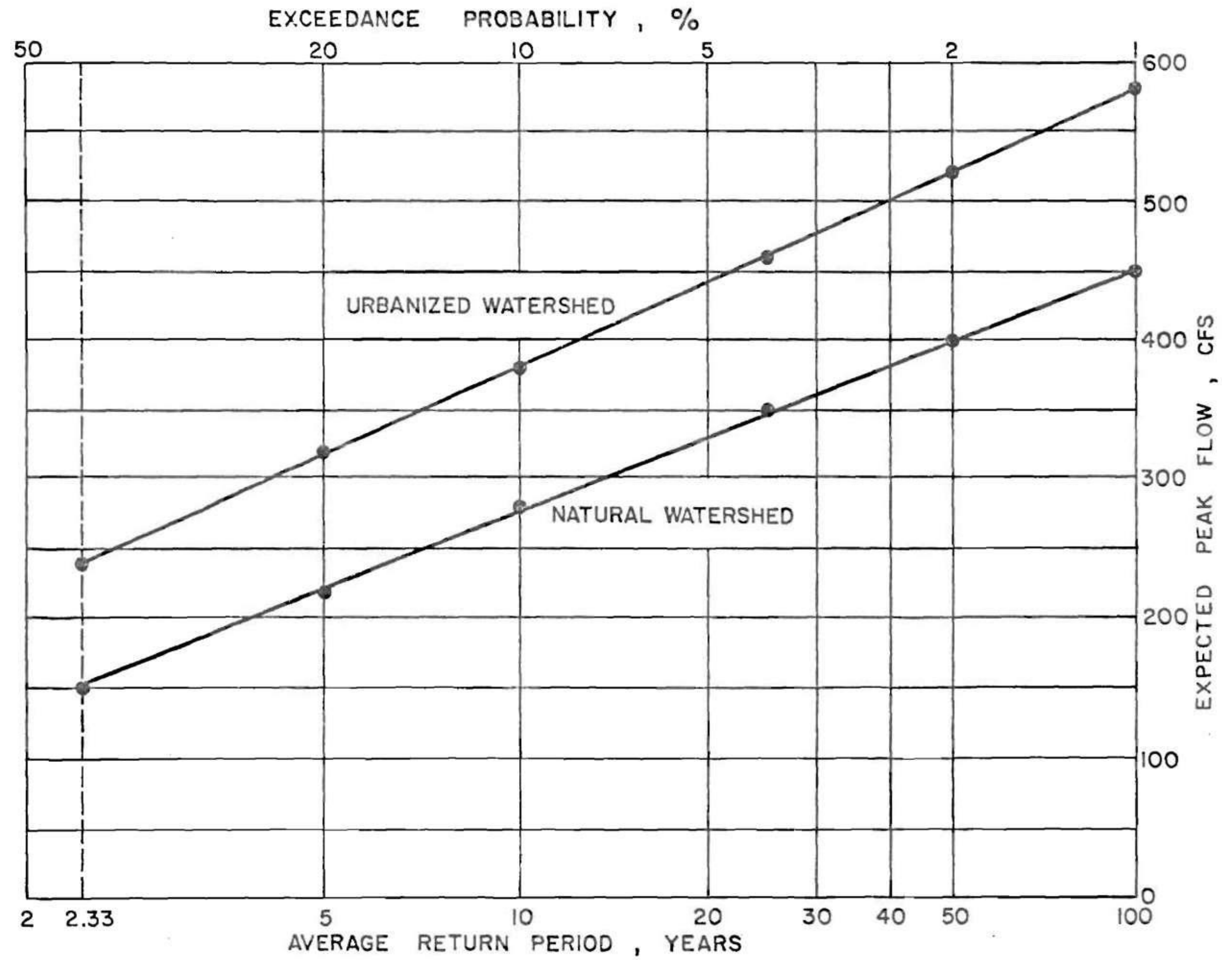

Figure B-11. Frequency-peak discharge Relationship for Example Problem Plotted on Gumbel Paper.

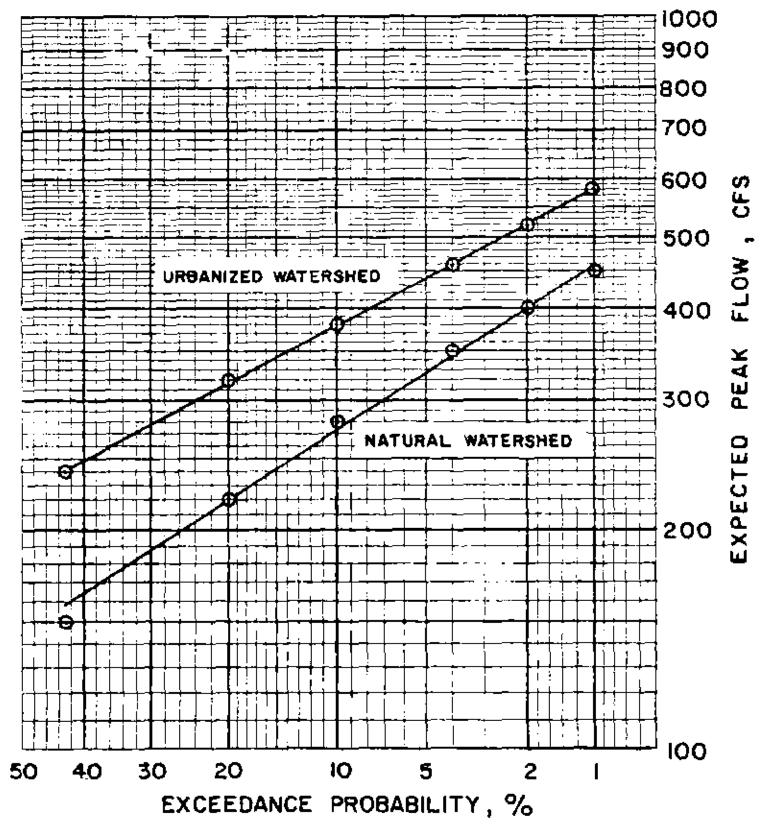

Figure B-12. Frequency-discharge Relationship for Example Problem Plotted on Log-Normal Probability Paper.

Table B-2. Summary of Calculations for Example Problem

Return Period, T	$\overline{Q}_{(n)}$	X $Q_{T(n)}:\overline{Q}_{(n)}$	= $Q_{T(n)}$	X U.F.	= $Q_{T(u)}$	Nominal $Q_{T(u)}$
2.33	153	1.00	153	1.54	236	240 CFS
5	153	1.45	222	1.42	315	320 CFS
10	153	1.82	278	1.37	381	380 CFS
25	153	2.29	350	1.32	462	460 CFS
50	153	2.63	402	1.30	523	520 CFS
100	153	2.97	454	1.28	581	580 CFS

T = average return period in years

$\overline{Q}_{(n)}$ = expected mean annual peak flow for natural watershed conditions

$Q_{T(n)}$ = expected peak flow for natural watershed conditions for return period, T

U.F. = Urbanization Factor

$Q_{T(u)}$ = expected peak flow for urbanized watershed conditions for return period, T

*This Appendix is an excerpt from "Determination of the Effects of Urbanization on Expected Peak Flows from Small Watersheds in DeKalb County, Georgia" by K. Randell Jones, Master of Science in Civil Engineering Thesis, Georgia Institute of Technology, June, 1978.

APPENDIX C*

ALTERNATE METHOD FOR DETERMINING EXPECTED PEAK FLOWS

An alternate method to that presented in Appendix B for determining expected peak flows has also been developed. It requires the separate determination of the effects of urbanization on the mean and standard deviation of the annual peak flow series. With this information and the selection of a probability density function, expected peak flows for selected return periods can be estimated.

The mean and standard deviation of the annual peak flow series can be determined directly from Equations C-1 and C-2 (Table C-1) or from a graphical method involving the following steps which are illustrated by an example:

Example Data

Drainage Area (A) = 150 acres

Percent Imperviousness (I) = 25%

Road Density (RD) = 10 miles per square mile

Percent of Channels Hydraulically Modified (HM) = 20%

Procedure

Step 1. Given the drainage area, use Figure C-1 to determine the mean and standard deviation of the annual peak flow series for an undeveloped watershed.

Example: $\overline{Q}_{(n)}$ = 153 CFS, $SD_{(n)}$ = 97 CFS

Step 2. Using Figure C-2 for the mean and Figure C-3 for the standard

Table C-1. Regression Equations for Estimating Expected Peak Flows (Alternate Method)

$$\overline{Q} = 1.319(A)^{0.949}\left[(1+I)^{1.657}\right]^{\left[\log(1+I)\right]^{0.1}}(1+RD)^{.044}(1+HM)^{0.116} \quad \text{(B-1)}$$

$$SD = 0.811(A)^{0.956}\left[(1+I)^{2.890}\right]^{\left[\log(1+I)\right]^{0.9}}(1+HM)^{0.124}(1+RD)^{0.020} \quad \text{(B-2)}$$

$\overline{Q}$ = expected mean annual peak flow in CFS
SD = standard deviation of the annual peak flow series in CFS
A = drainage area in acres
I = decimal fraction of impervious area
RD = road density in miles per square mile
HM = decimal fraction of channel length hydraulically modified

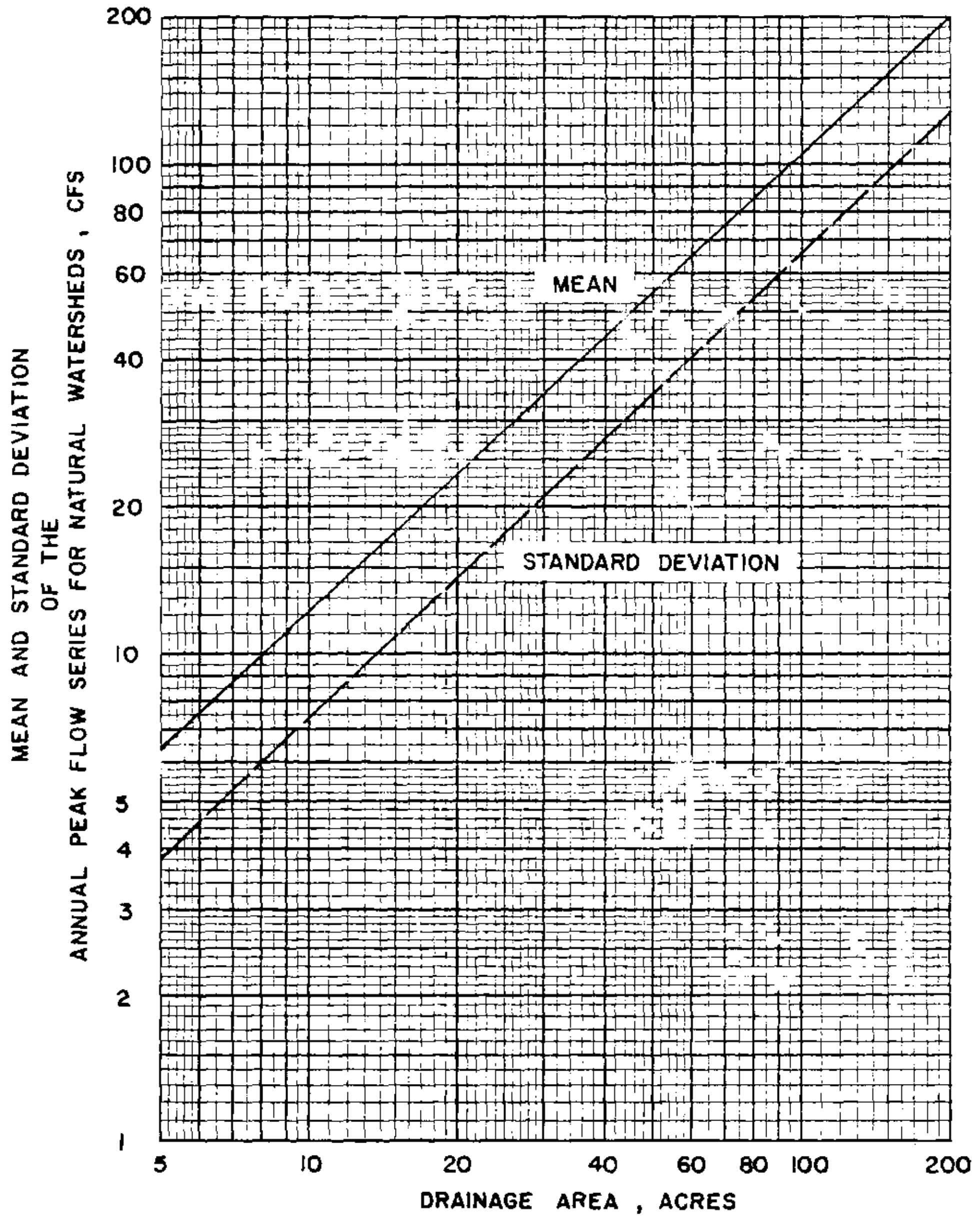

Figure C-1. Mean and Standard Deviation of the Annual Peak Flow Series for Natural Watersheds.

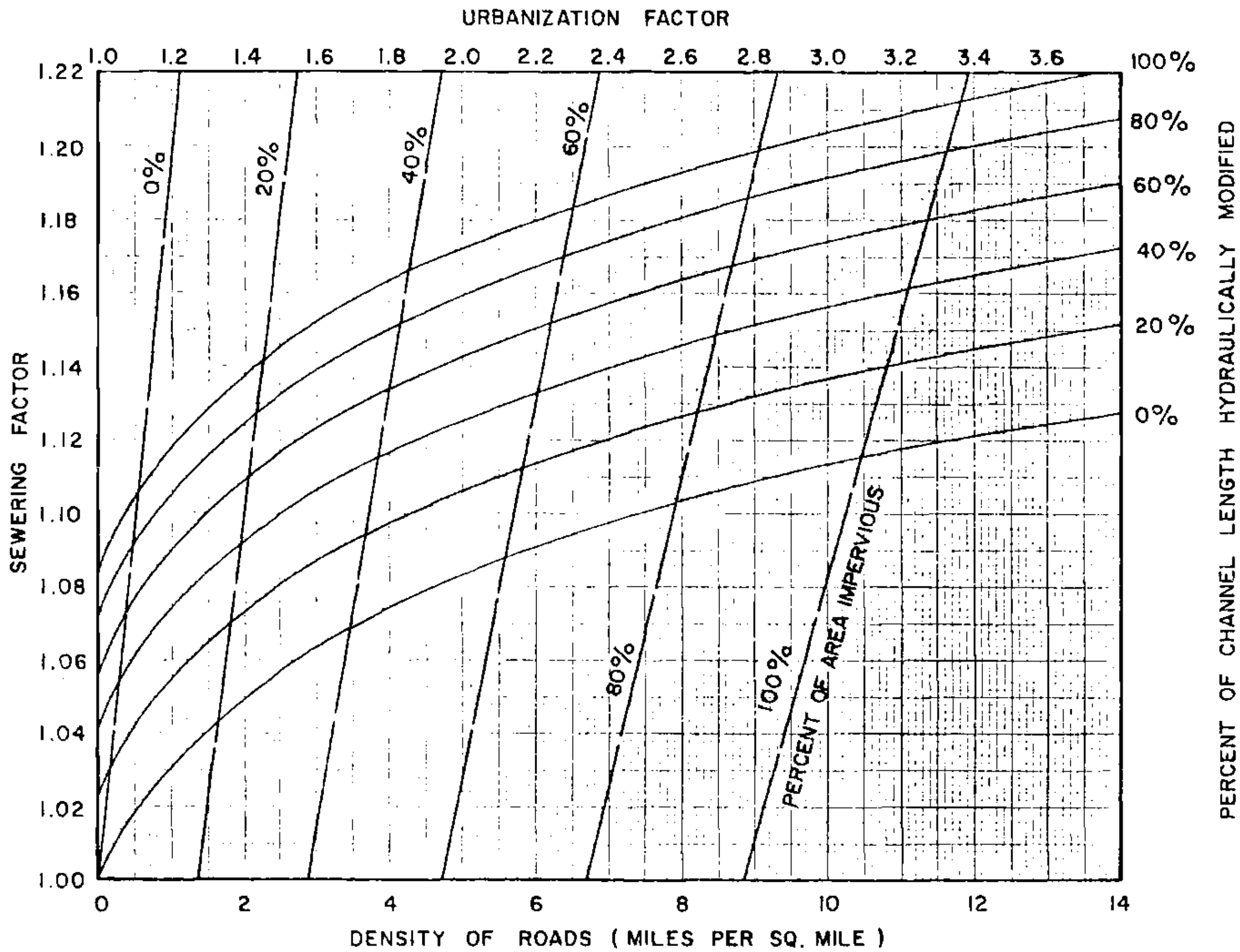

Figure C-2. Urbanization Factor Chart for the Expected Mean of the Annual Peak Flow Series.

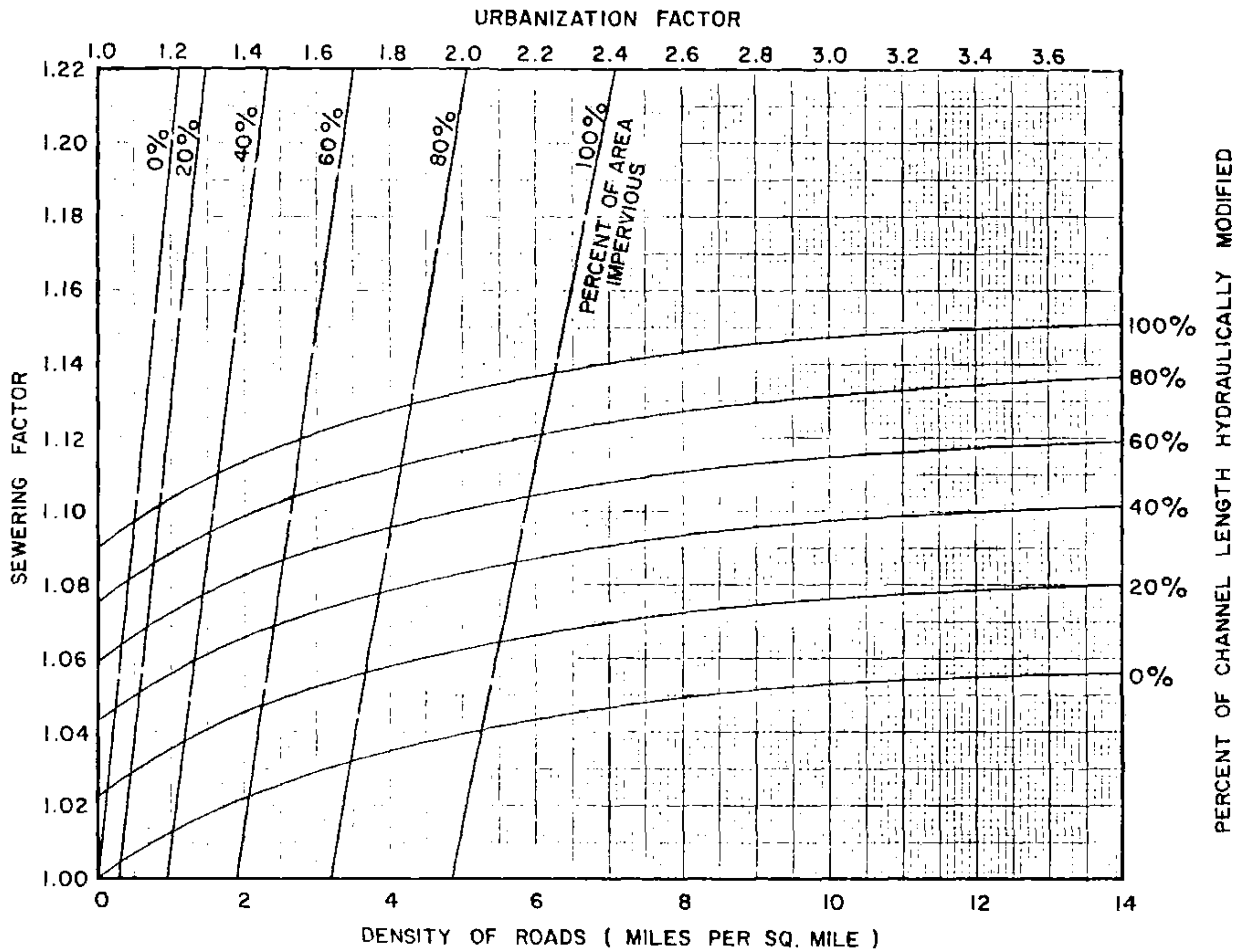

Figure C-3. Urbanization Factor Chart for the Standard Deviation of the Annual Peak Flow Series.

deviation, determine the Urbanization Factor using the same procedures outlined previously.

Example: $UF_{(mean)} = 1.54$, $UF_{(SD)} = 1.18$

Step 3. Multiply the results from Steps 1 and 2 to determine the mean and standard deviation of the annual peak flow series for the developed watershed.

Example: $\bar{Q}_{(u)} = 153 \times 1.54 = 236$ CFS

$SD_{(u)} = 97 \times 1.18 = 114$ CFS

Step 4. With the urbanized mean and standard deviation, expected peak flows can be estimated for a given probability density function. For the Gumbel or extreme value Type 1 distribution, the expected flows can be estimated from the equation

$$Q_T = \bar{Q} + (K_T \times SD) \tag{B-3}$$

where Q_T is the expected peak flow for return period T, $\bar{Q}$ is the mean of the annual peak flow series, K_T is a multiplier associated with the Gumbel distribution, and SD is the standard deviation of the annual peak flow series. Values of K_T are given in Table C-2.

Example: $\bar{Q}_{(u)} = 236$ CFS (say 240)

$Q_{5(u)} = 236 + 0.72(114) = 318$ CFS (say 320)

$Q_{10(u)} = 236 + 1.31(114) = 384$ CFS (say 390)

$Q_{25(u)} = 236 + 2.05(114) = 470$ CFS (say 470)

$Q_{50(u)} = 236 + 2.60(114) = 532$ CFS (say 530)

$Q_{100(u)} = 236 + 3.14(114) = 594$ CFS (say 590)

Note that these estimates are within 2 percent of those generated by the previously outlined simple method.

Table C-2. Multipliers for Extreme Value Distribution (After Lumb)

Probability of Exceedance	Return Period, T (years)	K_T
0.5	2	-0.16
0.2	5	0.72
0.1	10	1.31
0.04	25	2.05
0.02	50	2.60
0.01	100	3.14
0.005	200	3.68

Source: UROS: Urban Flood Simulation Model, Part 1. Documentation and Users Manual, Georgia Institute of Technology, 1975.

*This Appendix is an excerpt from "Determination of the Effects of Urbanization on Expected Peak Flows from Small Watersheds in DeKalb County, Georgia" by K. Randell Jones, Master of Science in Civil Engineering Thesis, Georgia Institute of Technology, June, 1978.

APPENDIX D

Table D. Expected Peak Rainfall Intensities in Inches per Hour as Generated from UROS Runoff File Soil 4 Using a Gumbel Distribution

Duration	Return Period, Years						
(Minutes)	2	2.33	5	10	25	50	100
5	4.25	4.56	5.95	7.08	8.52	9.58	10.63
10	3.66	3.93	5.13	6.10	7.33	8.23	9.14
15	3.15	3.39	4.49	5.38	6.52	7.35	8.18
20	2.84	3.07	4.11	4.94	6.01	6.79	7.57
25	2.59	2.81	3.75	4.52	5.50	6.21	6.93
30	2.34	2.53	3.39	4.09	4.97	5.62	6.27
40	1.95	2.11	2.80	3.36	4.08	4.61	5.13
50	1.66	1.78	2.33	2.78	3.35	3.77	4.18
60	1.44	1.54	2.00	2.37	2.84	3.19	3.54
80	1.16	1.24	1.61	1.90	2.27	2.55	2.82
100	0.99	1.05	1.34	1.58	1.87	2.09	2.31
125	0.84	0.89	1.12	1.30	1.54	1.71	1.88
150	0.73	0.77	0.97	1.13	1.33	1.48	1.63
200	0.58	0.61	0.77	0.89	1.05	1.16	1.28
250	0.49	0.51	0.64	0.74	0.86	0.96	1.05
300	0.43	0.45	0.55	0.64	0.74	0.82	0.90
400	0.34	0.36	0.44	0.51	0.59	0.65	0.71
500	0.29	0.30	0.37	0.42	0.49	0.54	0.59
600	0.25	0.26	0.32	0.37	0.43	0.47	0.51
750	0.22	0.23	0.28	0.32	0.37	0.41	0.45
900	0.19	0.20	0.24	0.28	0.33	0.36	0.40
1200	0.15	0.16	0.19	0.22	0.26	0.28	0.31
1500	0.13	0.13	0.16	0.18	0.21	0.23	0.26
1800	0.11	0.11	0.14	0.16	0.18	0.20	0.22

APPENDIX E

Table E. Ratios of Expected Peak Rainfall Intensity to Expected Mean Annual Peak Rainfall Intensity as Taken from UROS Runoff File Soil 4

Duration		Return Period, Years				
(Minutes)	2.33	5	10	25	50	100
5	1.00	1.30	1.55	1.87	2.10	2.33
10	"	1.30	1.55	1.86	2.09	2.32
15	"	1.32	1.59	1.92	2.17	2.41
20	"	1.34	1.61	1.95	2.21	2.46
25	"	1.34	1.61	1.96	2.21	2.47
30	"	1.34	1.61	1.96	2.22	2.47
40	"	1.33	1.60	1.94	2.19	2.44
50	"	1.31	1.56	1.88	2.11	2.34
60	"	1.30	1.54	1.84	2.07	2.30
80	"	1.30	1.53	1.83	2.05	2.27
100	"	1.27	1.50	1.78	1.[illegible]	2.20
125	"	1.26	1.46	1.73	1.[illegible]	2.11
150	"	1.25	1.46	1.72	1.[illegible]1	2.11
200	"	1.25	1.45	1.71	1.[illegible]9	2.08
250	"	1.24	1.44	1.67	1.[illegible]6	2.04
300	"	1.23	1.43	1.65	1.83	2.01
400	"	1.22	1.42	1.64	1.80	1.97
500	"	1.23	1.39	1.63	1.79	1.96
600	"	1.21	1.40	1.63	1.78	1.93
750	"	1.23	1.41	1.63	1.81	1.98
900	"	1.21	1.41	1.67	1.82	2.02
1200	"	1.21	1.40	1.66	1.78	1.98
1500	"	1.21	1.36	1.59	1.74	1.97
1800	"	1.25	1.43	1.61	1.78	1.96
Average	-	1.2688	1.4879	1.7638	1.9633	2.1721
Stand. Dev.	-	0.0466	0.0822	0.1288	0.1678	0.1953

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